VOL. XXXVIII. No. 5 MAY, 1960

THE STRUCTURAL ENGINEER

THE JOURNAL OF THE
INSTITUTION OF STRUCTURAL ENGINEERS

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The Elastic Lateral Stability of Trusses
by Dr. M. R. Horne

The Design of Slab Type Reinforced Concrete Stairways
by A. C. Liebenberg (Associate-Member)

The Design and Construction of the New Basic-Bessemer Plant at Port Talbot
Discussion on the Paper by J. W. P. Jaffé (Associate-Member)

Multi-Storey Car Parks
Discussion on the Paper by E. N. Underwood (Vice-President)

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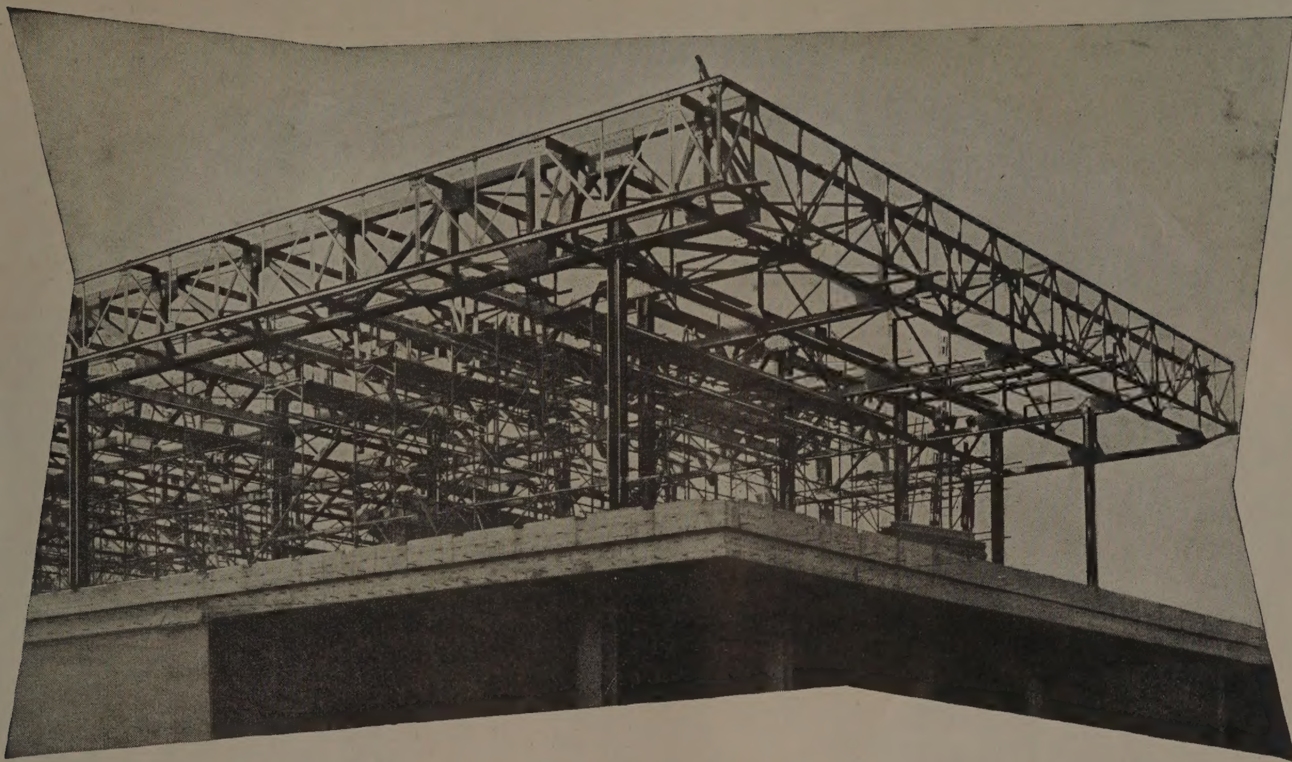


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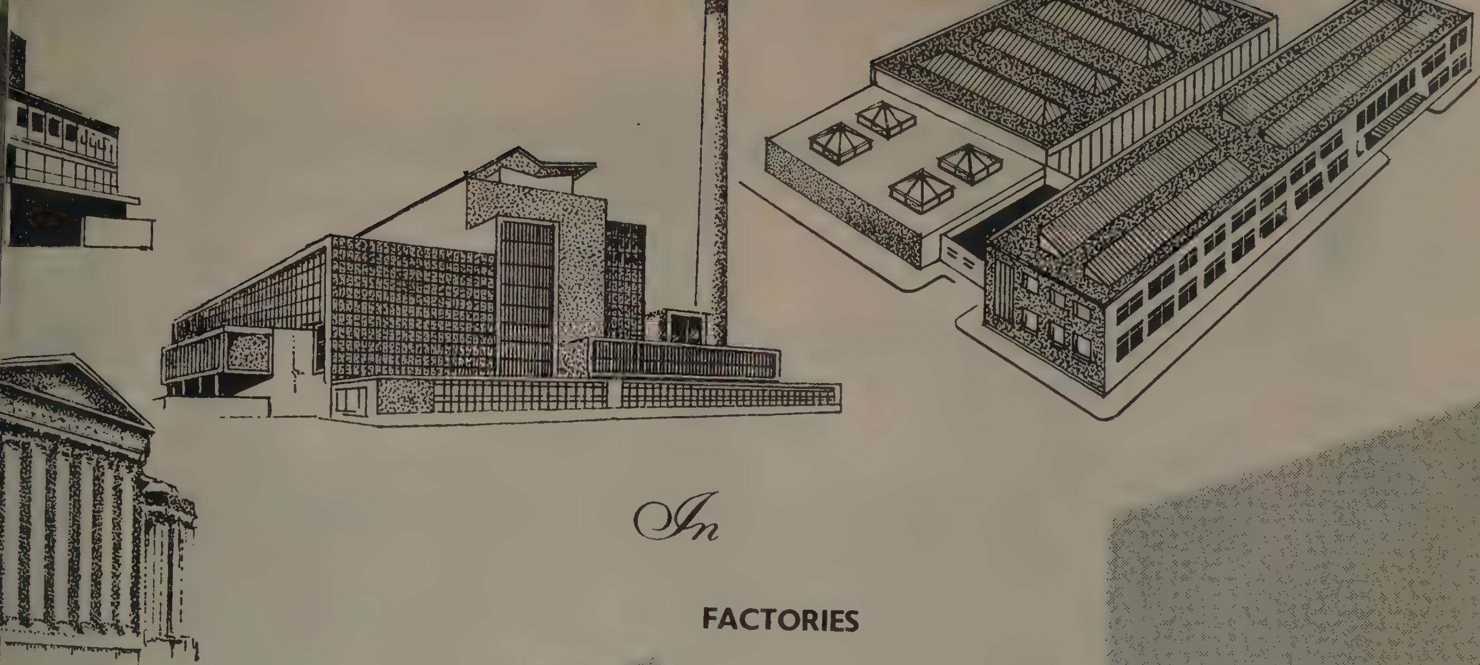

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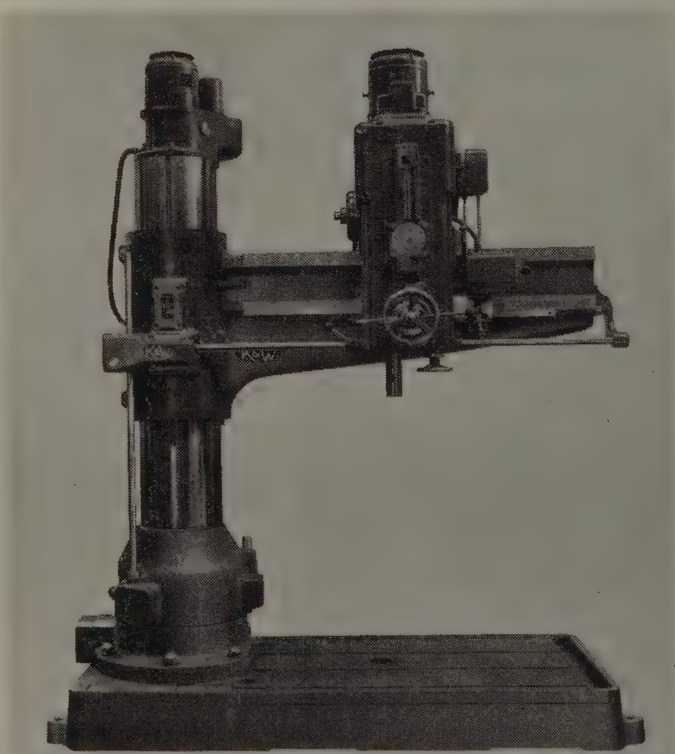

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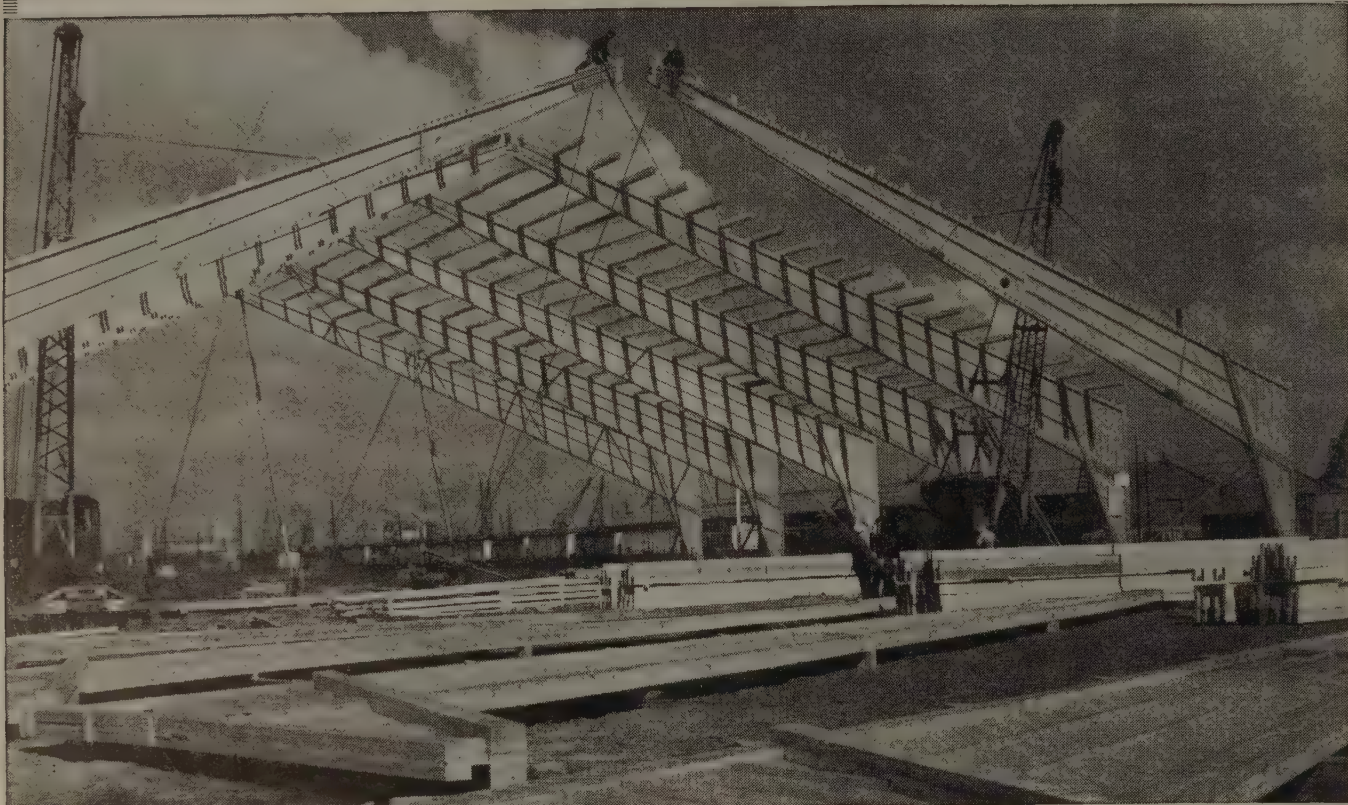

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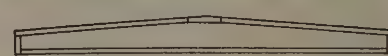

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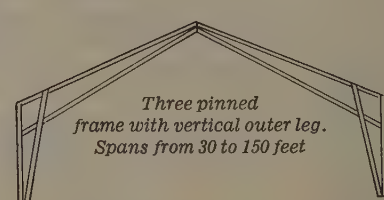

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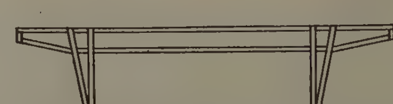

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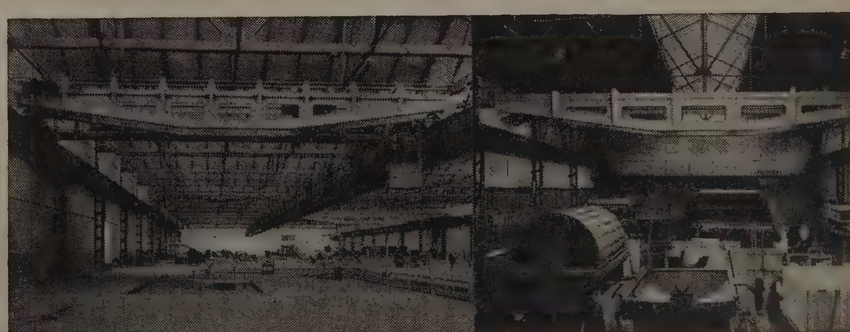

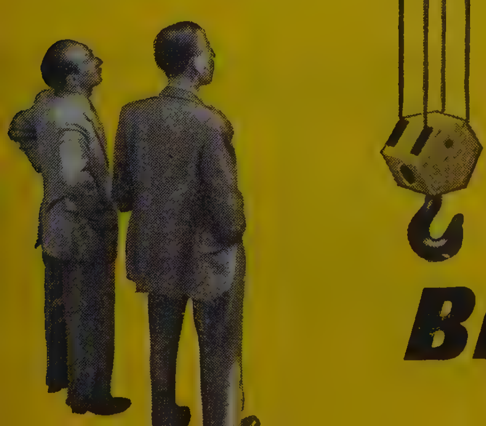

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CONTENTS

All Editorial communications should be addressed to The Editor, The Structural Engineer, 11, Upper Belgrave Street, S.W.1. All MSS. should be typewritten and algebraic signs and formulae be indicated distinctly.

Annual Subscription Rate to the Journal £3 3s. 0d. post free.

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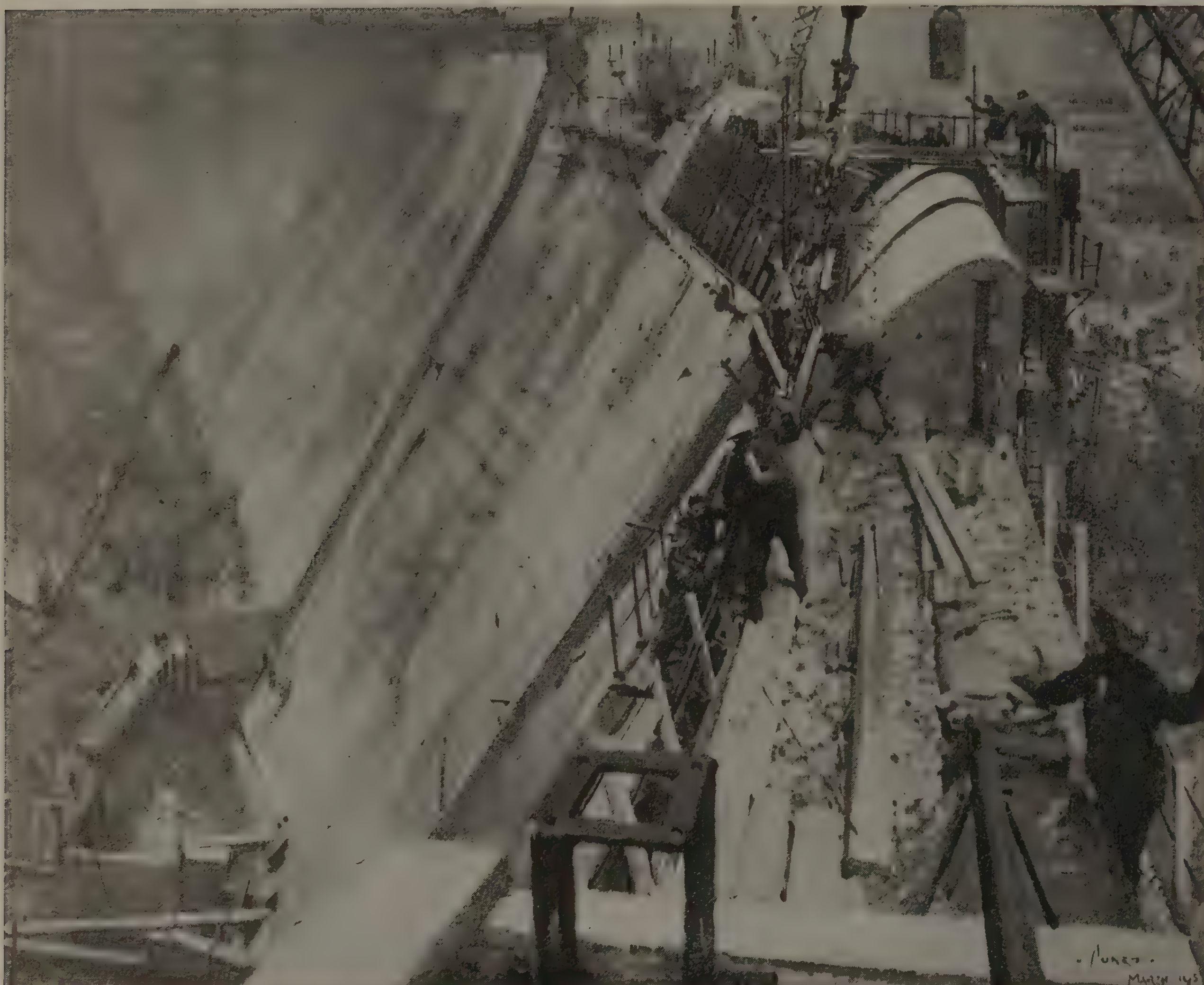


The Elastic Lateral Stability of Trusses

by M. R. Horne, Sc.D., A.M.I.C.E.

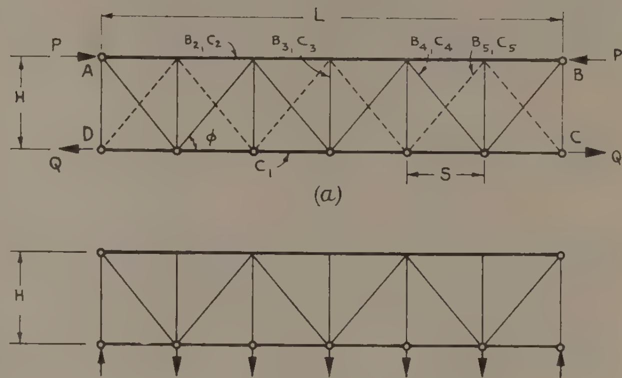
Synopsis

The article gives an approximate analysis, using the energy method, for the elastic stability out of their plane of trusses with outstanding compression chords, the tension chord being held in position. Previous discussions of this problem have usually considered it in terms of the stability of the compression chord as a member subjected to various elastic restraints. In the present treatment, the truss is considered as a whole, account being taken of the flexural and torsional rigidities of all the members, and of partial restraint of the tension chord against twisting. The analysis is interpreted in relation to a number of common arrangements of vertical and diagonal web members. The analysis is derived for a truss in which the compression chord is of uniform section, carrying a uniform thrust, but the use of some of the results as approximate criteria of stability for non-uniform conditions is also discussed.

Introduction

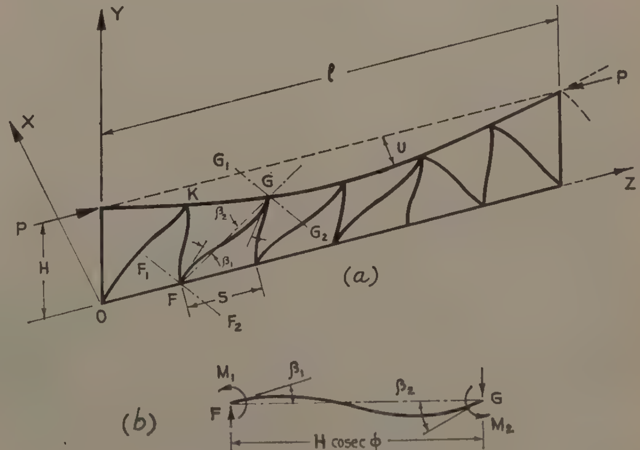
This article deals with the lateral stability of an elastic plane truss with parallel chords, such as that shown in Fig. 1(a). The analysis applies to any arrangement of vertical and diagonal web members, the only restriction being that no account is taken of intersections within the web. The truss is subjected to any combination of uniform bending moment and overall axial load. The compression chord (AB in Fig. 1(a)) sustains an axial load P and is laterally supported only by the web members of the truss, except at A and B, where it is assumed to be restrained against deflection out of the plane ABCD. The chord CD may carry any axial load Q (tensile or compressive), and is laterally supported at the panel points. Allowance is made in the analysis for elastic restraints against twisting of the chord CD about its longitudinal axis, these restraints being applied at panel points. Although this simplified form of loading is assumed, some of the results may be applied approximately to a truss more realistically loaded as shown in Fig. 1(b) provided the vertical loads are applied to the lower chord only, and the web members do not sustain compressive axial loads which are a high proportion of their axial loading capacities as pin-ended struts. The analysis is of particular interest in relation to trusses composed of tubular members, but is not restricted to such cases. The joints are all assumed to be rigid, and the members prismatic between panel points. For the sake of analysis it is also assumed that the chords are each of uniform section, while each set of web members (for example, the verticals) is of uniform section throughout the truss.

When the axial load P in the compression chord reaches a critical value, the truss buckles laterally as shown in Fig. 2(a). The buckling mode may be in a single half-wave, or in a series of almost equal half-waves. The compression chord is acted upon by a series of torques, moments and shear forces from the web members.

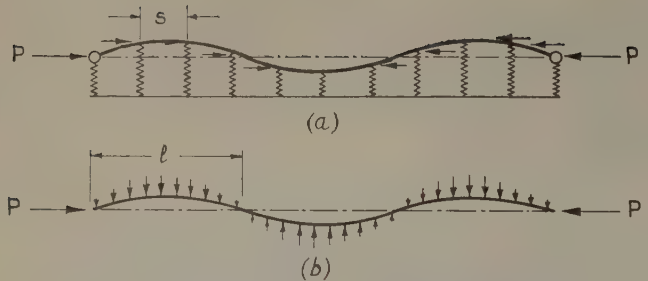


TRUSS UNDER (a) UNIFORM THRUST AND TENSION.
(b) VARYING BENDING MOMENT.
FIG. 1.

In most treatments of this problem, the stability of the compression chord is investigated by assuming that the restraints from the web members may be represented by the action of springs lying in a horizontal plane, as shown in Fig. 3(a). Bleich[1] suggests methods for solving this idealisation of the problem, allowing for changing axial load and cross-section of the compression chord and variations in the spacing and stiffness of the springs. For the simpler case of a uniform axial load in a chord of uniform section, with equal spacing and stiffness of the springs, an early approximate solution by Engesser[2] gives good results. If the springs in Fig. 3(a) require a force k to produce unit deflection, and the springs are



TRUSS IN THE BUCKLED STATE.
FIG. 2.



STABILITY OF ELASTICALLY SUPPORTED BAR.

FIG. 3.

TABLE 1

MEMBER	FLEXURAL RIGIDITY	TORSIONAL RIGIDITY.
TENSION BOOM	—	C_1
COMPRESSION BOOM	B_2	C_2
VERTICALS	B_3	C_3
DIAGONALS	B_4, B_5	C_4, C_5

$B = (B_4 + B_5) . \sin^3\phi$

$C = (C_4 + C_5) \sin^3\phi$

$A = \frac{H^2}{S} \times$ {FLEXURAL STIFFNESS OF TRANSVERSE MEMBERS AT EACH LOWER PANEL POINT.}

spaced at intervals s, Engesser replaces the springs by a continuous elastic medium which resists unit displacement by a force of $\frac{k}{s}$ per unit length of the chord, as shown in Fig. 3(b). It is then found that the chord buckles in half-waves of length l where

$$l = \pi\left(\frac{EIs}{k}\right)^{\frac{1}{4}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

in which EI is the flexural rigidity of the chord for bending out of the plane of the truss. The critical value of the axial load P is

$$P = 2\sqrt{\frac{EIk}{s}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Engesser showed experimentally that equation (2) is remarkably accurate provided $\frac{l}{s} > 1{\cdot}8$ and this has been confirmed theoretically by Bleich[1]. Equation (2) has been used extensively for bridge trusses in which the web members in Fig. (1) are almost completely direction fixed at their feet. The Engesser solution assumes that the chord is infinitely long and that the ends are fixed against lateral deflection with sufficient rigidity for the buckling load not to be reduced by buckling at the ends. The effect of completely free ends has been discussed by Zimmermann[3], while Chwalla[4] considers ends of finite rigidity. The case of chords of finite length in a continuous elastic medium, but with ends fixed against lateral deflection, has been discussed by Timoshenko[5], who deals not only with uniform axial loads in the chord, but also with axial loads varying parabolically.

Reference to Fig. 2 shows that the compression chord in a truss is not only restrained against lateral deflection (in direction OX), but that the chord members also provide restraint against the twisting of the chord about its longitudinal axis (parallel to OZ). The rotation of the chord at panel points about axes parallel to OY is also partially restrained, this restraint being due to twisting in the vertical chord members and to bending and twisting in the diagonal members. The buckling of a uniformly compressed, uniform chord member with equally spaced lateral and rotational restraints has been discussed by Budiansky *et al*[6]. For the compression chord in Fig. 2, this treatment would allow for restraints at panel points against deflections parallel to OX and for restraint against rotation about axes parallel to OY, but would ignore the effects associated with the twisting of the chord about its own longitudinal axis. The effect of bending moment in the vertical members, and their interaction with the twisting of the chord has been discussed by Hrennikoff[7] who, however, ignores some of the other restraints.

An exhaustive study of the problem of buckling of chords in pony trusses has recently been conducted by Holt[8]. It will be appreciated that a complete solution is extremely involved, and Holt's treatment is too complicated for practical use. In a review of theoretical and experimental work on the buckling of top chords in pony trusses, Handa[9] comes to the conclusion that the simple formula of Engesser gives results as good as any for the collapse loads of actual trusses, and that the complications of most treatments are not worthwhile. This is perhaps not surprising in view of Bleich's findings in relation to Engesser's formula. The trusses considered by Handa are composed of open section members, so that effects associated with the resistance of the chord and web members to twisting are of no importance provided local torsional buckling does not occur. When closed sections are used, and buckling rigidities become of the same order as flexural rigidities, the Engesser formula can no longer be expected to give reasonable results. The essential features of the Engesser solution may, however, be retained by considering all restraints to the chord from the web members to be distributed continuously as in a special sort of elastic medium. It is thus possible to make a sufficiently accurate allowance for all the restraints which have been considered in a more elaborate manner by the many authors who have written on the subject. This generalised solution to the buckling of a compression chord in a truss is the subject of the present paper.

It is a common feature of the treatments previously given that they consider the feet of the web members to be completely or partially restrained, without any allowance for the behaviour of the tension chord. Because of the high rotational restraint at the feet of the web members in bridge trusses, the neglect of the tension chord is justified, but trusses forming part of a building structure may be in a different category. Subsidiary members such as sheeting rails, connected to the tension chord, will usually suffice to restrain it in position laterally, but may not have large enough flexural stiffness to offer significant restraint against twisting. If the tension chord is of tubular section, it will then contribute appreciably to the stability of the truss. A solution which allows for the resistance to twisting of the tension chord must necessarily treat the truss as a whole, and this is the basis of what follows. The analysis is based on the energy method, allowance being made for the resistance to bending and twisting of all the members, and also for partial twisting restraint applied to the tension chord. The analytical results are summarised in Table 2 on pages 150 and 151. The solutions are interpreted specifically in relation to six arrangements of vertical and diagonal members, but are not restricted to these arrangements.

Notation

The notation is summarised in Figs. 1(a) and 2 and in Table 1. The flexural rigidities B_2, B_3, B_4 and B_5 are the values of EI for bending out of the plane of the truss (E modulus of elasticity, I moment of inertia). The torsional rigidities C_1 to C_5 are the values of GJ (G elastic shear modulus and J the St. Venant torsion constant). The flexural stiffness of the members (not shown) which restrain the tension chord CD against twisting is defined by the quantity A, which has the dimensions of the flexural and torsional rigidities ((force) $\times$ (distance)^2). If the external members attached to the tension chord produce a torque of T at each panel point for unit angle of twist of the chord, then $A = \frac{H^2}{s} T$ where H is the depth between centres of chords and s is the distance between panel points (Fig. 1(a)). The symbol $\varnothing$ denotes the angle between the diagonal members and the tension and compression chords. The length of the truss, between points at which the compression chord is restrained against lateral displacement, is denoted by L, while l is the half-wavelength for the compression chord in the buckled state. The thrust in the compression chord is denoted by P.

Various symbols, (μ, η, B, C, D, F) are used to denote functions of the above quantities, and are defined as occasion arises in Table 2. In the analysis we take axis OZ along the centre of the tension chord (Fig. 2(a)), OY in the plane of the truss perpendicular to OZ, and OX perpendicular to OY and OZ. The angle of twist of the tension chord is denoted by θ, and of the compression chord by θ_2, while the deflection of a point on the compression chord out of the plane OYZ is denoted by u. Taking any particular web member FG in the deformed state (Fig. 2(b)), the angles between the tangents at F and G and the straight line FG are denoted by β_1 and β_2 respectively.

General Analysis

It will be assumed that, in the buckled state, the lateral deflection of the compression chord (in direction XO) is given by

$$u = H\theta \sin\frac{\pi z}{l} \qquad (3)$$

where z is measured from one end. It is assumed that the tension chord remains straight. These deflected forms neglect the local distortions produced in the chords by the bending and twisting resistance of the web members, and are justified provided the flexural rigidities of the chords are large compared with the flexural and torsional rigidities of the web members. The angles of twist of the tension and compression chords (clockwise about OZ) are represented by

$$\theta_1 = a_1\theta \sin\frac{\pi z}{l}, \qquad (4)$$

$$\theta_2 = a_2\theta \sin\frac{\pi z}{l}. \qquad (5)$$

The external work, per length l of the truss, due to the thrust P in the compression chord is U_P where

$$U_P = \frac{P}{2}\int_0^l \left(\frac{du}{dz}\right)^2 dz = \frac{\pi^2}{4}\frac{PH^2}{l}\theta^2. \qquad (6)$$

Since the tension chord is assumed to remain straight no work is done by the force Q. The external work U_P has to be equated to the total strain energy due to buckling in the members of the truss.

The tension chord has strain energy due to twisting U_{C1} where

$$U_{C1} = \frac{C_1}{2}\int_0^l \left(\frac{d\theta_1}{dz}\right)^2 dz = \frac{\pi^2 a_1^2}{4}\frac{C_1}{l}\theta^2. \qquad (7)$$

The compression chord has strain energies U_{B2} due to bending and U_{C2} due to twisting where

$$U_{B2} = \frac{B_2}{2}\int_0^l \left(\frac{d^2u}{dz^2}\right)^2 dz = \frac{\pi^4}{4}\frac{B_2H^2}{l^3}\theta^2, \qquad (8)$$

$$U_{C2} = \frac{C_2}{2}\int_0^l \left(\frac{d\theta_2}{dz}\right)^2 dz = \frac{\pi^2 a_2^2}{4}\frac{C_2}{l}\theta^2. \qquad (9)$$

We consider now the twisting and flexure of any diagonal member FG (Fig. 2). The line F_1F_2 is taken through F perpendicular to FG and in the plane OYZ. The line G_1G_2 passes through G and is parallel to F_1F_2. Since there is continuity between the tension chord and diagonal FG at F, the tangent to FG rotates during buckling about F_1F_2 through the angle $\left(a_1\theta \sin\frac{\pi z}{l} . \sin\varnothing\right)$ where z is the distance of F from the origin. Similarly, the tangent to FG at G rotates about G_1G_2 through the angle

$$\left(a_2\theta \sin\frac{\pi(z+s)}{l} . \sin\varnothing + \frac{\pi H}{l}\theta \cos\frac{\pi(z+s)}{l} . \cos\varnothing\right)$$

The chord FG rotates about F_1F_2 through the angle $\left(\theta \sin\frac{\pi(z+s)}{l} . \sin\varnothing\right)$. The angles β_1 and β_2 in Fig. 2(b) are thus

$$\beta_1 = \theta\left[\sin\frac{\pi(z+s)}{l} - a_1 \sin\frac{\pi z}{l}\right]\sin\varnothing,$$

$$\beta_2 = \theta\left[\left(\sin\frac{\pi(z+s)}{l} - a_2\sin\frac{\pi(z+s)}{l}\right)\sin\varnothing - \frac{\pi H}{l}\cos\frac{\pi(z+s)}{l} . \cos\varnothing\right].$$

We now make the assumption that H and s are small compared with l, and ignore s in comparison with z. Hence

$$\beta_1 = (1 - a_1)\,\theta \sin\frac{\pi z}{l} . \sin\varnothing, \qquad (10)$$

$$\beta_2 = (1 - a_2)\,\theta \sin\frac{\pi z}{l} . \sin\varnothing - \frac{\pi H}{l}\theta \cos\frac{\pi z}{l} . \cos\varnothing. \qquad (11)$$

If we denote the bending moments at F and G in the member FG by M_1 and M_2 respectively as shown in Fig. 2(b), then the slope-deflection equations give

$$M_1 = \frac{2B_4}{H}(2\beta_1 + \beta_2)\sin\varnothing, \qquad (12)$$

$$M_2 = \frac{2B_4}{H}(\beta_1 + 2\beta_2)\sin\varnothing \qquad (13)$$

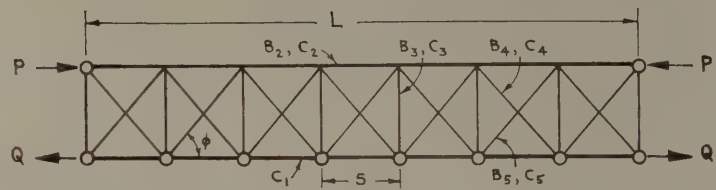
where B_4 is the flexural rigidity of FG. The flexural energy in FG is this $\triangle U_{B4}$ where

$$\triangle U_{B4} = \tfrac{1}{2}(M_1\beta_1 + M_2\beta_2) = \frac{2B_4}{H}(\beta_1^2 + \beta_1\beta_2 + \beta_2^2)\sin\varnothing. \qquad (14)$$

TABLE 2.

$B = (B_4 + B_5)\sin^3\phi$

$C = (C_4 + C_5)\sin^3\phi$



$B = 2B_4\sin^3\phi$

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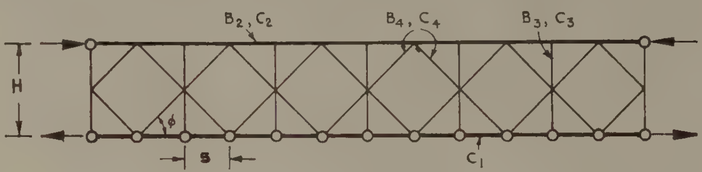
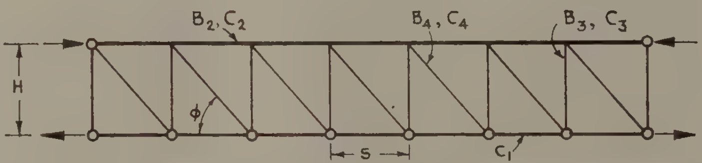


$B = B_4\sin^3\phi$

$C = C_4\sin^3\phi$



O DENOTES COMPLETE RESTRAINT AGAINST LATERAL DEFLECTION.

GENERAL SOLUTION.

$D = C\cot\phi + (B+B_3)\tan\phi, \qquad F = C\cot\phi + 4(B+B_3)\tan\phi.$

LET $n = \left(\frac{L}{m\pi H}\right)$ WHERE $m = 1, 2, \text{----------},$

$$a_1 = \frac{6(B+B_3)\left(2D + \frac{C_2}{n^2}\right)}{\left(A+F+\frac{C_1}{n^2}\right)\left(F+\frac{C_2}{n^2}\right) + 9C(B+B_3) - DF},$$

$$a_2 = \frac{6(B+B_3)\left(A + 2D + \frac{C_1}{n^2}\right)}{\left(A+F+\frac{C_1}{n^2}\right)\left(F+\frac{C_2}{n^2}\right) + 9C(B+B_3) - DF},$$

$$P = \frac{1}{H^2}\Bigg[\frac{B_2}{n^2} + a_1^2 C_1 + a_2^2 C_2 + 4B\cot\phi + (C+C_3)\tan\phi + n^2\left\{a_1^2 A + 4\left[(1-a_1)^2 + (1-a_1)(1-a_2) + (1-a_2)^2\right](B+B_3)\tan\phi + (a_1-a_2)^2 C\cot\phi\right\}\Bigg].$$

TAKE SMALLEST P ($m = 1, 2, \text{-------}$).

$A = 0$, $C_1 = C_2$

$\mu^2 = 12\tan\phi.$

$\frac{\mu}{2}\frac{\sqrt{B_2(B+B_3)}}{C_1} < 1$ AND $\left(\frac{L}{H}\right)^2 > \frac{\pi^2}{\mu}\sqrt{\frac{B_2}{B+B_3}}\;\frac{1}{1-\frac{\mu}{2}\frac{\sqrt{B_2(B+B_3)}}{C_1}},$

$$P = \frac{1}{H^2}\left[2\mu\sqrt{B_2(B+B_3)}\left\{1 - \frac{\mu}{4}\frac{\sqrt{B_2(B+B_3)}}{C_1}\right\} + 4B\cot\phi + (C+C_3)\tan\phi\right].$$

$\frac{\mu}{2}\frac{\sqrt{B_2(B+B_3)}}{C_1} < 1$ AND $\left(\frac{L}{H}\right)^2 < \frac{\pi}{\mu}\sqrt{\frac{B_2}{B+B_3}}\;\frac{1}{1-\frac{\mu}{2}\frac{\sqrt{B_2(B+B_3)}}{C_1}}$ *

OR $\frac{\mu}{2}\frac{\sqrt{B_2(B+B_3)}}{C_1} > 1$ (ALL VALUES OF L),

$$P = \pi^2\frac{B_2}{L^2} + \frac{1}{H^2}\left[\frac{2C_1}{1+\frac{2\pi^2}{\mu^2}\left(\frac{H}{L}\right)^2\frac{C_1}{B+B_3}} + 4B\cot\phi + (C+C_3)\tan\phi\right].$$

SAFE RESULT, ALL LENGTHS, $\frac{\mu}{2}\frac{\sqrt{B_2(B+B_3)}}{C_1} < 1,$

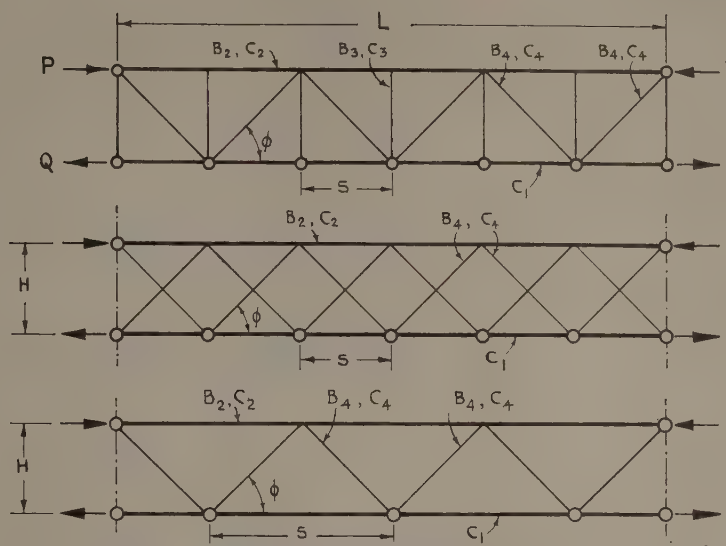
$$P = \frac{1}{H^2}\left[2\mu\sqrt{B_2(B+B_3)}\left\{1 - \frac{\mu}{4}\frac{\sqrt{B_2(B+B_3)}}{C_1}\right\} + 4B\cot\phi + (C+C_3)\tan\phi\right].$$

SAFE RESULT, ALL LENGTHS, $\frac{\mu}{2}\frac{\sqrt{B_2(B+B_3)}}{C_1} > 1,$

$$P = \frac{1}{H^2}\left[2C_1 + 4B\cot\phi + (C+C_3)\tan\phi\right].$$

* For $\frac{\pi}{\mu}$ read $\frac{\pi^2}{\mu}$

TABLE 2 CONTINUED.



$B = B_4 \sin^3\phi$
$C = C_4 \sin^3\phi$

$B = 2B_4 \sin^3\phi, \; B_3 = 0$
$C = 2C_4 \sin^3\phi, \; C_3 = 0$

$B = B_4 \sin^3\phi, \; B_3 = 0$
$C = C_4 \sin^3\phi, \; C_3 = 0$

T = FLEXURAL STIFFNESS OF TRANSVERSE ATTACHMENTS AT EACH LOWER PANEL POINT. $A = \frac{H^2}{5}T.$

$\eta^2 = 12\tan\phi\left\{\frac{C\cot\phi + (B+B_3)\tan\phi}{C\cot\phi + 4(B+B_3)\tan\phi}\right\}.$	$A = 0$, $C_1 = 0$
$\eta\frac{\sqrt{B_2(B+B_3)}}{C_2} < 1$ AND $\left(\frac{L}{H}\right)^2 > \frac{\pi^2}{3}\sqrt{\frac{B_2}{B+B_3}}\;\frac{1}{1-\eta\frac{\sqrt{B_2(B+B_3)}}{C_2}}$, $P = \frac{1}{H^2}\left[2\eta\sqrt{B_2(B+B_3)}\left\{1-\frac{\eta}{2}\frac{\sqrt{B_2(B+B_3)}}{C_2}\right\} + 4B\cot\phi + (C+C_3)\tan\phi\right].$	
† $\eta\frac{\sqrt{B_2(B+B_3)}}{C_2} < 1$ AND $\left(\frac{L}{H}\right)^2 < \frac{\pi^2}{3}\sqrt{\frac{B_2}{B+B_3}}\;\frac{1}{1-\eta\frac{\sqrt{B_2(B+B_3)}}{C_2}}$, OR $\eta\frac{\sqrt{B_2(B+B_3)}}{C_2} > 1$ (ALL VALUES OF L), $P = \pi^2\frac{B_2}{L^2} + \frac{1}{H^2}\left[\frac{C_2}{1+\frac{\pi}{\eta^2}\left(\frac{H}{L}\right)^2\cdot\frac{C_2}{B+B_3}} + 4B\cot\phi + (C+C_3)\tan\phi\right].$	
SAFE RESULT, ALL LENGTHS, $\eta\frac{\sqrt{B_2(B+B_3)}}{C_2} < 1$, $P = \frac{1}{H^2}\left[2\eta\sqrt{B_2(B+B_3)}\left\{1-\frac{\eta}{2}\frac{\sqrt{B_2(B+B_3)}}{C_2}\right\} + 4B\cot\phi + (C+C_3)\tan\phi\right].$	
SAFE RESULT, ALL LENGTHS, $\eta\frac{\sqrt{B_2(B+B_3)}}{C_2} > 1$, $P = \frac{1}{H^2}\left[C_2 + 4B\cot\phi + (C+C_3)\tan\phi\right].$	
AS $\left\{\begin{matrix} A = 0 \\ C_1 = 0 \end{matrix}\right\}$ WITH C_2 REPLACED BY C_1.	$A = 0$, $C_2 = 0$
$\eta^2 = 12\tan\phi\left\{\frac{C\cot\phi + (B+B_3)\tan\phi}{C\cot\phi + 4(B+B_3)\tan\phi}\right\}.$	$C_1 = 0$, $C_2 = 0$
$\left(\frac{L}{H}\right)^2 > \frac{\pi^2}{3}\sqrt{\frac{B_2}{B+B_3}}\sqrt{1+\eta^2\frac{B+B_3}{A}}$, $P = \frac{1}{H^2}\left[\frac{2\eta\sqrt{B_2(B+B_3)}}{\sqrt{1+\eta^2\frac{B+B_3}{A}}} + 4B\cot\phi + (C+C_3)\tan\phi\right].$	
$\left(\frac{L}{H}\right)^2 < \frac{\pi^2}{3}\sqrt{\frac{B_2}{B+B_3}}\sqrt{1+\eta^2\frac{B+B_3}{A}}$, $P = \pi^2\frac{B_2}{L^2} + \frac{1}{H^2}\left[\frac{\frac{\eta^2}{\pi^2}\left(\frac{L}{H}\right)^2(B+B_3)}{1+\eta^2\frac{B+B_3}{A}} + 4B\cot\phi + (C+C_3)\tan\phi\right].$	
SAFE RESULT, ALL LENGTHS, $P = \frac{1}{H^2}\left[\frac{2\eta\sqrt{B_2(B+B_3)}}{\sqrt{1+\eta^2\frac{B+B_3}{A}}} + 4B\cot\phi + (C+C_3)\tan\phi\right].$	

† For $\frac{\pi}{\eta^2}$ read $\frac{\pi^2}{\eta^2}$

If there is one diagonal number for each length s of the truss (as in Fig. 2), the mean bending energy in the diagonals per unit length of the truss is

$$\frac{\Delta U_{B4}}{s} = \frac{\Delta U_{B4}}{H} \tan \varnothing .$$

The total bending energy in the diagonals throughout the length l is

$$U_{B4} = \int_0^l \left(\frac{\Delta U_{B4} \tan \varnothing}{H} \right) dz. \quad . \quad . \quad . \quad (15)$$

Substituting for β_1 and β_2 from equations (10) and (11) in equation (14), and thence for ΔU_{B4} in equation (15), the value of U_{B4} becomes, on integration,

$$U_{B4} = \left[\{(1-a_1)^2 + (1-a_1)(1-a_2) + (1-a_2)^2\} \tan \varnothing + \left(\frac{\pi H}{l}\right)^2 \cot \varnothing \right] \frac{(B_4 \sin^3 \varnothing) l}{H^2} \cdot \theta^2. \quad . \quad (16)$$

The end F of the diagonal FG twists about FG through the angle $\left(a_1\, \theta \sin \frac{\pi z}{l} . \cos \varnothing \right)$, while end G twists through the angle

$$\left(a_2\, \theta \sin \frac{\pi(z+s)}{l} . \cos \varnothing - \frac{\pi H}{l} \theta \cos \frac{\pi(z+s)}{l} . \sin \varnothing \right)$$

Again, neglecting s in comparison with z, the twisting energy ΔU_{C4} in the diagonal is

$$\Delta U_{C4} = \frac{C_4}{2H} \left\{ (a_1 - a_2)\, \theta \sin \frac{\pi z}{l} \cdot \cos \varnothing + \frac{\pi H}{l}\, \theta \cos \frac{\pi z}{l} \cdot \sin \varnothing \right\}^2 \sin \varnothing . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

The total twisting energy in the diagonals is U_{C4} where

$$U_{C4} = \int_0^l \left(\frac{\Delta U_{C4} \tan \varnothing}{H} \right) dz, \quad . \quad . \quad . \quad (18)$$

whence

$$U_{C4} = \left[(a_1 - a_2)^2 \cot \varnothing + \left(\frac{\pi H}{l}\right)^2 \tan \varnothing \right] \times \frac{(C_4 \sin^3 \varnothing) l}{4H^2} \theta^2. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

The bending and twisting energies in the vertical members of the truss may be derived from the values for the diagonal members as follows. It is assumed that the vertical members are spaced at intervals of $s = H \cot \varnothing$, as in Fig. 2(a). The angles β_1 and β_2 for any vertical FK are obtained by putting $\varnothing = \frac{\pi}{2}$ in equations (10) and (11), and the bending energy ΔU_{B3} is then obtained from equation (14) by putting $\varnothing = \frac{\pi}{2}$ and replacing B_4 by B_3. The total bending energy U_{B3} in all the verticals is obtained from equation (15), but $\tan \varnothing$ must here be retained since it corresponds to $\frac{H}{s}$ and defines the spacing of the members.

Hence

$$U_{B3} = \{(1-a_1)^2 + (1-a_1)(1-a_2) + (1-a_2)^2\} \times \tan \varnothing . \frac{B_3 l}{H^2} \theta^2. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (20)$$

Similarly we obtain the twisting energy U_{C3} in the verticals,

$$U_{C3} = \left(\frac{\pi H}{l}\right)^2 \tan \varnothing . \frac{C_3 l}{4H^2} \theta^2. \quad . \quad . \quad . \quad . \quad . \quad . \quad (21)$$

Finally we require the energy stored in the subsidiary members which restrain the tension chord against twisting. The restraining torque per unit length is $\frac{T}{s} a_1\, \theta \sin \frac{\pi z}{l}$, and since $\frac{T}{s} = \frac{A}{H^2}$, the total restraining energy U_A over the buckling length l is

$$U_A = \frac{A}{2H^2} \int_0^l \left(a_1\, \theta \sin \frac{\pi z}{l} \right)^2 dz = a_1^2 \frac{Al}{4H^2} \theta^2. \quad . \quad (22)$$

The energy equation for the length l of the truss is

$$U_P = U_A + U_{B2} + U_{B3} + U_{B4} + U_{C1} + U_{C2} + U_{C3} + U_{C4}. \quad . \quad . \quad (23)$$

Upon substituting for the expressions for the separate terms, we have

$$PH^2 = a_1^2 C_1 + a_2^2 C_2 + 4B \cot \varnothing + (C + C_3) \tan \varnothing + \frac{B_2}{n^2} + n^2 a_1^2 A + n^2 (a_1 - a_2)^2 C \cot \varnothing + 4n^2[(1-a_1)^2 + (1-a_1)(1-a_2 + (1-a_2)^2](B + B_3) \tan \varnothing$$

where . . . . (24)

$$n = \frac{l}{\pi H},$$
$$B = B_4 \sin^3 \varnothing,$$
$$C = C_4 \sin^3 \varnothing.$$

The buckling length l may assume any value $\frac{L}{m}$ where m is a digit. For each l, the corresponding buckling load P is obtained by putting $\frac{\partial P}{\partial a_1} = 0$ and $\frac{\partial P}{\partial a_2} = 0$, so that P is a minimum with respect to the arbitrarily chosen coefficients a_1 and a_2.

Hence

$$2(3 - 2a_1 - a_2)(B + B_3) \tan \varnothing - (a_1 - a_2)\, C \cot \varnothing - a_1 A = \frac{a_1 C_1}{n^2}, \quad . \quad . \quad (25)$$

$$2(3 - a_1 - 2a_2)(B + B_3) \tan \varnothing + (a_1 - a_2)\, C \cot \varnothing = \frac{a_2 C_2}{n^2}. \quad . \quad . \quad . \quad (26)$$

Equations (25) and (26) may be solved for a_1 and a_2 and the resulting values of a_1 and a_2 substituted in equation (24). If $P = P_1$ corresponds to $m = 1$, $P = P_2$ to $m = 2$ etc., it will usually be most convenient first to calculate P_1 and P_2; if $P_1 < P_2$, then P_1 is the required solution. If $P_1 > P_2$, then P_3 is calculated; if then $P_2 < P_3$, P_2 is the solution, and so on. The complete equations for this general solution are given at the beginning of Table 2 on page 150. The diagrams at the head of the table show how the results may be applied to trusses with some common arrangements of web members by suitably adjusting the significance of the terms B and C.

The above procedure is lengthy, and in some cases a more convenient solution, on the safe side, may be obtained if it is assumed that l may possess any value less than L. We then have $\frac{\partial P}{\partial l} = O$ or, since $n = \frac{l}{\pi H}$, $\frac{\partial P}{\partial n} = O$.

Hence

$$a_1^2\, A + (a_1 - a_2)^2\, C \cot \varnothing + 4[(1 - a_1)^2 + (1 - a_1)(1 - a_2) + (1 - a_2)^2]\,(B + B_3) \tan \varnothing = \frac{B_2}{n^4} \quad \cdot \quad \cdot \quad \cdot \quad (27)$$

The value of P now appears as the solution of equations (24) to (27), provided $l < L$ or $n < \frac{L}{\pi H}$. The general solution for P is not explicit, but an expression for P may be obtained for the following special cases.

(1) $A = O, \quad C_1 = C_2,$
(2) $A = O, \quad C_1 = O,$
(3) $A = O, \quad C_2 = O,$
(4) $C_1 = O, \quad C_2 = O.$

Results for these four sets of conditions are summarised in Table 2. Their derivation will be illustrated by considering the second case, $A = O, \quad C_1 = O$.

The Case A = 0 C₁ = 0

If we put $\gamma = \left(\frac{B + B_3}{C}\right) \tan^2 \varnothing,$

equation (25) becomes

$$(1 - a_1) = \frac{1 - 2\gamma}{1 + 4\gamma}(1 - a_2) \quad \cdot \quad \cdot \quad \cdot \quad (28)$$

and if $\eta^2 = 12 \tan \varnothing \frac{1 + \gamma}{1 + 4\gamma}$, equations (26) and (27) become respectively

$$n^2 \eta^2 \frac{(1 - a_2)}{a_2} = \frac{C_2}{B + B_3}, \quad \cdot \quad \cdot \quad \cdot \quad (29)$$

$$n^4 \eta^2 (1 - a_2)^2 = \frac{B_2}{B + B_3} \quad \cdot \quad \cdot \quad \cdot \quad (30)$$

Hence

$$a_2 = \frac{\eta \sqrt{B_2(B + B_3)}}{C_2}, \quad \cdot \quad \cdot \quad (31)$$

$$n^2 = \frac{1}{\eta}\sqrt{\frac{B_2}{B + B_3}} \quad \frac{1}{1 - \eta \frac{\sqrt{B_2(B + B_3)}}{C_2}} \quad \cdot \quad (32)$$

By substituting for a_1, a_2 and n in equation (24), we obtain

$$P = \frac{1}{H^2}\left[2\eta\sqrt{B_2(B + B_3)}\ \left\{1 - \frac{\eta}{2}\frac{\sqrt{B_2(B + B_3)}}{C_2}\right\} + 4\, B \cot \varnothing + (C + C_3) \tan \varnothing\right] \quad (33)$$

Equation (33) applies provided the length $l = n\pi H$ obtained from equation (32) is less than the length of the truss L, that is provided

$$\left(\frac{L}{H}\right)^2 > \frac{\pi^2}{\eta}\sqrt{\frac{B_2}{B + B_3}} \quad \frac{1}{1 - \eta \frac{\sqrt{B_2(B + B_3)}}{C_2}} \quad \cdot \quad (34)$$

When L is smaller than the limit given by the inequality (34), the buckling length l must be taken equal to L, and the value of P is obtained by substituting $n = \frac{L}{\pi H}$ in equations (24), (25) and (26). Hence it is found that

$$P = \pi^2 \frac{B_2}{L^2} + \frac{1}{H^2}\left[\frac{C_2}{1 + \frac{\pi^2}{\eta^2}\left(\frac{H}{L}\right)^2 \frac{C_2}{B + B_3}} + 4\, B \cot \varnothing + (C + C_3) \tan \varnothing\right] \quad \cdot \quad (35)$$

A safe solution for P will result if the member is assumed to be infinitely long, whatever its actual length may be. For such a safe solution, equation (33) is appropriate provided the value of n given by equation (32) is real, that is, provided

$$\eta \frac{\sqrt{B_2(B + B_3)}}{C_2} < 1.$$

When this condition is not satisfied, it denotes that the buckling length is infinitely long, and a safe value for P is obtained from equation (35) by putting $L = \infty$, that is

$$P = \frac{1}{H^2}\left[C_2 + 4B \cot \varnothing + (C + C_3) \tan \varnothing\right]. \quad (36)$$

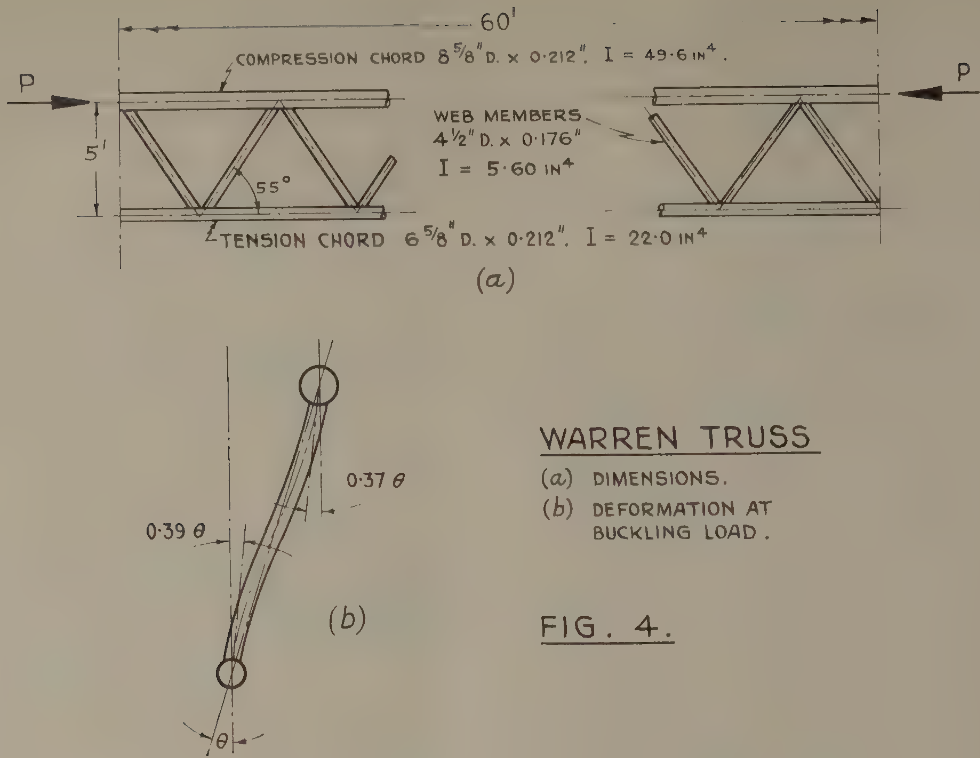
The interesting feature of equations (33) and (36) is that they provide a safe estimate of the buckling load of the truss without reference to the length between laterally restrained points. These safe estimates are given for all the special cases in Table 2.

Numerical Example

The Warren truss in Fig. 4(a), 60 feet long and 5 feet deep between chord centres, is composed of circular steel tubes of the sections indicated. The modulus of elasticity, E, is 13,000 tons/in.^{2} and the elastic shear modulus, G, is 5,000 tons/in.^{2} Each panel point on the lower chord is partially restrained against twisting about the longitudinal axis to give $A = 150 \times 10^3$ tons in^2., corresponding to a stiffness at each panel point of 3,500 tons ins.

The value of P which would cause instability, all members being assumed to remain elastic, may be estimated in various ways, and the solutions are summarised in Table 3. The first result quoted is based on the general solution in Table 2, it being found that three half-waves gives the minimum value of P (313 tons). With two half-waves, P = 364 tons, and with four half-waves, P = 347 tons. A cross-section of the truss in the deflected state when three half-waves form is shown in Fig. 4(b).

The analytical solutions given in Table 2 for the three cases ($A = O, C_1 = C_2$), ($A = O, C_1 = O$) and ($C_1 = O, C_2 = O$) have been used in lines 2 to 4 of Table 3 to provide safe estimates of P. The values of the various flexural and torsional stiffnesses assumed in each case are quoted, as also are the buckling lengths l. It is interesting to note that the actual buckling length (20 feet, line 1 of Table 3) is smaller than any of those obtained in the subsequent safe estimates of P, being slightly less than the shortest (26 feet, line 4). The shortest buckling length obtained from the safe estimates may, in fact, always be used



WARREN TRUSS

(a) DIMENSIONS.

(b) DEFORMATION AT BUCKLING LOAD.

FIG. 4.

TABLE 3.

	ANALYSIS USED.	VALUES OF FLEXURAL AND TORSIONAL RIGIDITIES TONS – IN.²						BUCKLING LENGTH l FEET.	CRITICAL P TONS.
		A $10^3\times$	B_2 $10^3\times$	B_4 $10^3\times$	C_1 $10^3\times$	C_2 $10^3\times$	C_4 $10^3\times$		
1	GENERAL	150	645	73	220	496	56	20	313
2	$A=0,\ C_1=C_2$	0	645	73	220	220	56	60	172
3	$A=0,\ C_1=0$	0	645	73	0	496	56	41	173
4	$C_1=0,\ C_2=0$	150	645	73	0	0	56	26	176
5	ENGESSER	150	645	73	0	0	0	26	133

to obtain an approximation to the actual buckling length. Finally, if the Engesser formula is applied (line 5), allowance being made for the incomplete torsional restraint on the lower chord, then $P = 133$ tons. The Engesser formula thus underestimates the critical load in this case by some 57 per cent.

Application to Trusses with Non-Uniform Thrust in Compression Chord

The analysis contained in this article has been derived by reference to the case of uniform thrust in the compression chord. The "safe" results contained in Table 2 may, however, be used to obtain an approximate criterion of safety for trusses with non-uniform thrusts, and also non-uniform cross-sections, since these "safe" results do not involve the length between points of support. A truss will be stable provided the thrust P does not exceed the value given by the appropriate "safe" equations in Table 2 at any section along the truss. The member properties assumed in any such calculation are those corresponding to the particular section of the truss considered, certain properties being reduced until one of the cases contained in Table 2 becomes relevant.

An exception to the validity of the above criterion may occur when the axial thrusts in the web members become appreciable in relation to their individual buckling loads as pin-ended members. Hrennikoff[7] in his treatment considered the effect of such thrusts, but this is a subject requiring more detailed study. It may be noted that in many trusses, the effect of axial thrusts in some of the web members will be largely compensated by equal tensions in adjacent members. Moreover, the most critical section of the truss for buckling of the compression chord will usually correspond to that section where the axial loads in the web members are small. It may therefore be concluded that the effect of axial thrusts in web members may usually be neglected.

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Discussion

The Council would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by the 30th July, 1960.

Book Reviews

Advanced Structural Design, by Cyril S. Benson. (London : Batsford, 1959). 9in. × 6in., 329 plus xiii pp. 50s.

This interesting and practical book consists essentially of the material one would compile in designing seventeen structures. Of these ten deal with structural steel projects, five with reinforced concrete structures and two with brickwork construction. The problems analysed are diverse : included are the complete designs for a grain silo, highway bridge, tank structure, 120 feet span shed, two-bay portal plant house, theatre balcony, multi-storey office building, bunkers, gantries and a chimney.

The work is clearly presented in a digestible form, and the calculations are skilfully augmented with explanation where necessary. It should help to broaden the structural horizons of the many designers who, unfortunately, are only trained in dealing adequately with one structural material and rarely get the chance to see what the other fellow does in practice.

Unfortunately, the book must lose a great deal of impact in that BS.449 : 1948 is used for the steelwork designs. It may also be said that the designs tend to be dated and many will regret that there is no reference to the Plastic Theory in steelwork, or to prestressed concrete and that little guidance is given to the student designing welded structures.

Despite these shortcomings, the book may prove a useful stimulant for students planning to take the Associate membership examination. R.H.

Civil Engineering Contracts Organization, by John C. Maxwell-Cook. (London : Cleaver-Hume Press, 1959). 8½ in. × 5½ in., 220 plus viii pp. 22s. 6d.

The Author sets out comprehensively the procedure from the inception of a scheme by the employer to its development in the contract stage, giving the relationships between the various parties concerned, advisory and executive, for the successful completion of a civil engineering project. A survey is given of the contract documents in general use with definitions of contract terms and useful comments on the clauses, and some piquant observations on financial arrangements.

The personnel required for the execution of the complete scheme is described from top level to the labourer and detailed as regards their duties and relationships. The importance to the client and contractor of a preliminary site survey and report by the consulting engineer on ground conditions has not, perhaps, been given sufficient place.

The chapters on specifications are perhaps not sufficiently up to date particularly regarding concrete which is mostly specified by quality control and strength nowadays, nor is there any mention of materials testing. Most large contracts are usually now equipped with a site laboratory for this purpose.

The site organization section contains many useful suggestions and aids to economy and smooth running of work on a construction site, and stresses the desire for design to be allied to easy and rapid execution at site. The Glossary forms a welcome appendix.

The book will be interesting and informative to all concerned in the development and execution of civil and structural engineering projects. F.T.B.

Linear Structural Analysis, by P. B. Morice, D.Sc., Ph.D., A.M.I.C.E., A.M.I.Struct.E. (London : Thames & Hudson, 1959). 9½ in. × 6¼ in, 170 plus xii pp., 35s.

Since the 1930's, methods such as strain-energy and least work have steadily been giving way to methods of successive approximation for the analysis of many forms of elastic structures. Quite recently, however, the introduction of matrix algebra to structural analysis has greatly increased the usefulness of the classical approach. Moreover, electronic computers are available to perform the tedious numerical work involved in processes such as matrix inversion, thus making possible the solution of problems having a high degree of indeterminacy.

This book forms an excellent introduction to the subject. The first two chapters are largely devoted to the basic concepts of strain energy, Castigliano's theorems, and influence coefficients, and include a variety of examples. There follows a treatment of the question of the degree of indeterminacy of structures by a method which is stated to be without exception in its application to skeletal structures and, as such, represents a notable advance on previous methods.

Matrix algebra is then introduced, particular attention being paid to computational procedures, the description of each type of matrix operation being accompanied by a simple worked example. The following chapters deal with scale factors, transformation of co-ordinate systems and a number of points concerning release systems (of which the suitable choice may simplify the analysis). The final chapter deals very briefly with the programming of an electronic digital computer for structural analysis, and an appendix gives four worked examples which nicely illustrate the methods developed in the main text. There are numerous references which will help the student who wishes to pursue further any aspect of the subject.

E. M.

The Design of Slab Type Reinforced Concrete Stairways

by A. C. Liebenberg, B.Sc.(Eng.), A.M.I.Struct.E., A.M.I.C.E.

Synopsis

The Author develops a method of design of slab type stairways, based on deductions made from full scale and scale model tests, which incorporates the extensional stiffness produced by the interaction of the stair flights and landings and which has been successfully applied in practice.

Although the very simple analytical procedure does not yield 'exact' solutions the inaccuracies involved appear to be small for most cases occurring in practice. Not only does the method lead to economy in design but the architecture can in many cases be enhanced by the elimination of beams and columns without the use of excessively thick sections, which would be required if the design procedures used in normal practice were employed.

Introduction

The normal practice in the design of slab type stairs, as typified by the recommendations of the "B.S. Code of Practice for Reinforced Concrete No. 114, 1957, Clauses 341 and 342," is to neglect the extensional stiffness provided by the interaction of the stair flights and landings and to consider the total resistance to the loading to be provided by bending forces only. Full scale load tests[1] carried out on a stairway of a reinforced concrete building in Johannesburg in 1952 demonstrated clearly that the normal method of analysis gives an incorrect picture of the load carrying mechanism in many types of stairs and leads to incorrect and unnecessarily conservative designs.

The conditions necessary for total or partial extensional stiffness to be produced exist in many types of stairways occurring in practice. As it would, however, be difficult to lay down general rules covering all the possible variations it is proposed to develop the analytical method for a few typical cases from which the general design procedure will be obvious. The Author has applied this method of analysis to the design of several types of stairs which have been constructed with complete success. Various Authors have in recent papers[2] described stairs designed along similar lines, but the basic approach to the problem as presented here is new.

Analysis

An 'exact' analysis of slab type stairs in accordance with the theory of elasticity is beyond the scope of existing rigorous mathematical methods. One of the more powerful numerical methods would have to be applied. Such an amount of work could not be justified in practice for a structure of this type. The method developed below is only an approximate method, but compares favourably with normal design procedures used in the design of reinforced concrete slab and beam systems in which compatibility of strains and deflections is mostly neglected and similar arbitrary assumptions as regards load and stress distribution are made. These and other so-called secondary effects which cannot be accurately determined on account of the many uncertainties inherent in reinforced concrete can usually be ignored because they are included in the factor of safety.

As the thickness of the slab elements of stairs is usually small compared with the planar dimensions, the extensional stiffness may, under certain conditions, greatly exceed the flexural stiffness. (The ratio is not equivalent to that of shell structures but is nevertheless significant). The *primary load carrying system* in such stairs is therefore produced by extensional (also known as membrane or planar) forces caused by the interaction of the stair flights, the landings and other external restraints such as walls, columns, beams and floor slabs. These 'points' or 'lines' of intersection of the slab elements become in effect 'supports' to *the secondary load carrying system* of bending forces in the slab elements, due to the fact that the resultant components of the extensional forces at these intersection points provide reactions which balance the shear forces in the slab elements.

The tests referred to above indicated that the deflections of the 'supports' at the intersection lines were small compared with the deflections due to bending of the stair flights and were of the same order of magnitude as the deformation of the brick wall supports. These 'supports' are therefore taken into account in the determination of the bending forces in the slab elements, the same assumptions being made as in normal design practice for the design of slab and beam systems.

The primary system may be *complete* in that it consists only of extensional forces or it may be an *incomplete* primary system which in order to have stability requires bending stiffness in certain members to supplement the incomplete extensional force system. It is essential to stress the necessity of such systems being completely stable for all possible modes of loading with sufficiently stiff external restraints to provide the reactions to the unbalanced internal extensional forces.

The sub-division of the analysis as described above is for convenience only. The relative importance of the two systems will depend on many factors including the type of stairs, the thickness of the slab elements in relation to the planar dimensions and the relative stiffnesses of the external restraints. It is seldom possible to make an accurate assessment of all these factors but as will be explained later, great accuracy is not required.

The analytical procedure can be summarised as follows :—

(a) Ascertain whether the conditions necessary for a primary system to function are present and determine the type of primary system and the effective 'supports' provided thereby to the secondary bending force system and any local direct forces that will result from any applied loading acting at an inclination to the axis of any member.

(b) Provide imaginary external restraints at the above-mentioned 'supports' so as to prevent displacement but allowing free rotation.

(c) Determine the magnitude of the secondary bending forces and local direct forces acting in the members due to the applied loading.

(d) Determine the resultant reactions on the imaginary 'supports'.

(e) Determine the magnitude of the forces in the primary system due to forces equal but opposite to these reactions. The actual stresses induced will be due to the combined effect of the forces acting in both systems.

Case I :—

Consider the stairs shown diagrammatically in Fig. 1(a). This is a simple case of a triangular arch and will serve well as an illustration of the method of analysis. The conditions necessary for a complete primary system to function are present. For the purposes of explanation an applied line loading acting as shown will be considered. Any other loading could be dealt with in a similar manner.

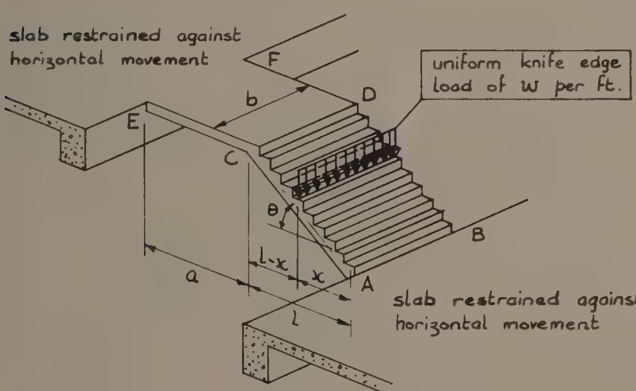

fig. 1(a) CASE 1

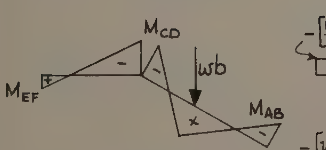

fig. 1(b) Bending Moments

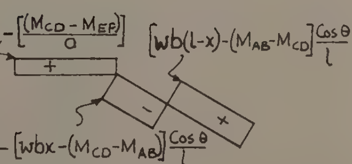

fig. 1(c) Shear Forces

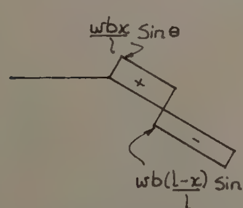

fig. 1(d) Local Direct Forces

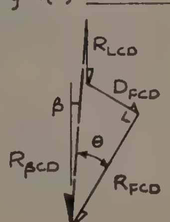

fig. 1(e) Force Diagram.

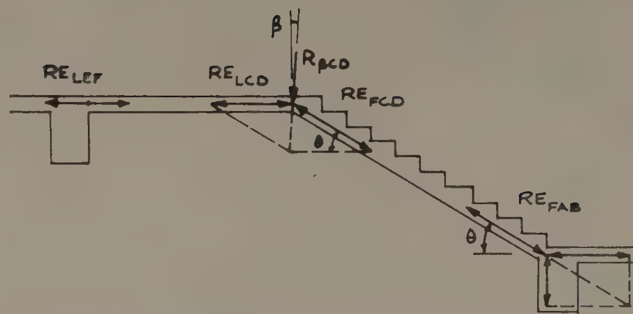

fig. 1(f) Resultant Extensional Forces.

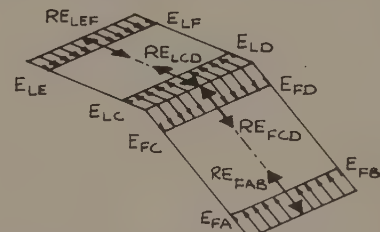

fig. 1(g) Extensional Forces

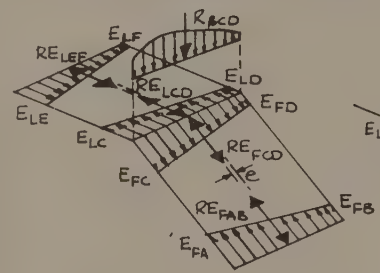

fig. 1(h).

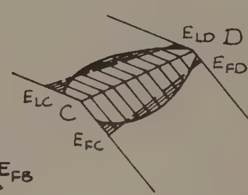

fig. 1(k).

In addition to the supports AB and EF the intersection line CD acts as a 'support' to the secondary bending force system. The actual bending forces will depend on the conditions of restraint at AB and EF and can be determined approximately by methods similar to those used in normal practice for the analysis of continuous slab systems.

For the bending forces and local direct forces shown in Figs. 1(b) to 1(d) the resultant reaction $R_{\beta CD}$ at CD acting at an angle β with the vertical will consist of three components as shown in Fig. 1(e), viz. :—

(i) $R_{FCD} = (wbx - [M_{CD} - M_{AB}])\,\dfrac{\cos\theta}{l}$

due to the bending forces in the flight and acting at an angle θ with the vertical.

(ii) A vertical reaction $R_{LCD} = \dfrac{-[M_{CD} - M_{EF}]}{a}$

due to the bending forces in the landing and

(iii) a reaction $D_{FCD} = \dfrac{wbx}{l}\sin\theta$

due to the local direct force in the flight.

Consequently the extensional force in the landing

$RE_{LCD} = RE_{LEF}$ [see Fig. 1(f)].

$= - R_{\beta CD}\,(\sin\beta + \cos\beta\,.\,\cot\theta)$

(Tensile forces are assumed to be positive).

If $M_{CD} - M_{AB} = 0$ then $\beta = 0$ and the resultant reaction will act vertically and will be equal to

$$R_{\beta CD} = \frac{wbx}{l} - \frac{(M_{CD} - M_{EF})}{a} \quad \ldots \quad (\beta = 0)$$

The extensional forces in the flight (including the local direct forces) are :—

$$RE'_{FCD} = - R_{\beta CD} \cdot \frac{\cos\beta}{\sin\theta} + \frac{wbx}{l} \sin\theta$$

and

$$RE'_{FAB} = - R_{\beta CD} \frac{\cos\beta}{\sin\theta} - \frac{wb\,(l - x)}{l} \sin\theta$$

(See notation on page 163).

Neglecting edge effects the extensional forces per unit width or the stresses can be determined approximately. In this particular case the extensional forces will be uniformly distributed across the width as shown in Fig. 1(g).

For a non-uniform loading the distribution of the resultant reaction $R_{\beta CD}$ at CD may also be non-uniform and non-linear as shown in Fig. 1(h).

In such a case the shape of the extensional stress distribution in the landing slab and stair flight at the intersection line CD will actually approximate to that of the reaction $R_{\beta CD}$. It is, however, sufficiently accurate to assume that the Bernouilli-Euler straight line stress distribution is applicable. In comparison with the bending stresses, the extensional stresses are in most cases small so that a considerable error in the analysis may not affect the final value significantly. As a result of the above assumptions there may be unbalanced forces acting at the intersection line CD. Although the effect of these forces is usually negligible some allowance should be made in the case of heavy concentrated loads acting at or near the line CD.

Although a precise solution of this problem is not possible by simple methods, it can be dealt with as described below.

For a distribution of the resultant reactions due to bending forces at CD as shown in Fig. 1(k), the unbalanced forces will be as shown shaded. These unbalanced forces will be resisted by a combination of extensional and bending forces. For a case like this where the intersection line CD is continuous over the full width, the extensional forces will resist almost the total of the unbalanced forces and the secondary bending forces can be safely neglected. The unbalanced extensional forces therefore have to be 'dissipated' through a certain length of slab which procedure will give a distribution of stress approximating more closely to the actual.

This problem is not unlike that of the deep beam. An approximate solution can be found by assuming the dissipation of the unbalanced forces to occur over a length equal to b. The stress produced thereby can be determined by the approximate method suggested by Magnel[3] for end blocks in prestressed concrete beams wherein he assumed a 45° spread of direct forces and that the stress diagram due to the bending moment acting in the plane of the slabs is a parabola of the third degree, the stress being zero at a distance b from CD. It will very rarely be necessary in practice to make any allowance for this effect.

If the resultant $R\beta_{CD}$ acts at a distance 'e' off the centre line of the flight the extensional force in the landing at C, assuming tensile forces to be positive will be

$$E_{LC} = + \frac{RE_{LCD}}{b}\left(1 + \frac{6e}{b}\right)$$

$$= - \frac{R_{\beta CD}\,(\sin\beta + \cos\beta.\cot\theta)}{b}\left(1 + \frac{6e}{b}\right)$$

and at D will be

$$E_{LD} = + \frac{RE_{LCD}}{b}\left(1 - \frac{6e}{b}\right)$$

$$= - \frac{R_{\beta CD}\,(\sin\beta + \cos\beta.\cot\theta)}{b}\left(1 - \frac{6e}{b}\right)$$

and similarly for the other forces.

Any movement of the supports AB and EF will cause CD to deflect and will therefore affect the magnitudes of the bending forces which in turn will determine the magnitude of the extensional forces. Provided the characteristics of the supports are known the resultant internal forces could be calculated but this would involve more work than is justified for such a small structure. A conservative estimate could however be made by covering the range of possible solutions. The problem is not unlike that of normal plane slabs supported on beams, walls and columns where a considerable relative deflection of supports may occur—an everyday problem for the practising designer. Bearing in mind the latest developments of the plastic and ultimate load or loadfactor methods of design any errors involved in the above procedure cannot be serious.

Any type of loading on the stairs e.g. partially distributed loading or point and line loads such as a balustrade wall along the outer edges, could be treated in a similar manner, the only complication being the calculation of the bending forces which is in no way different to the problems arising in the design of slabs supported on walls or beams.

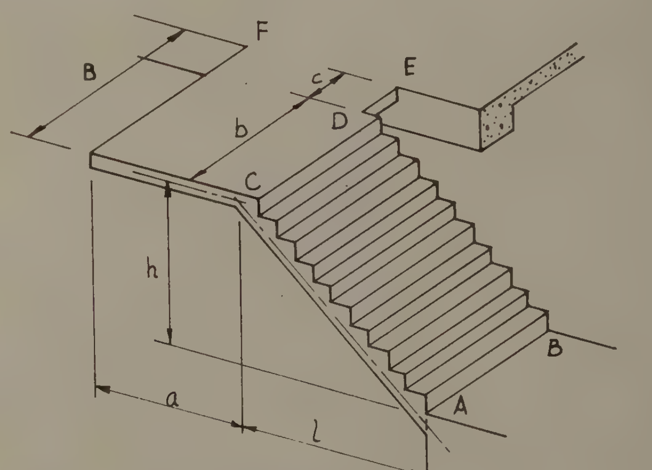

fig 2 (a) CASE II.

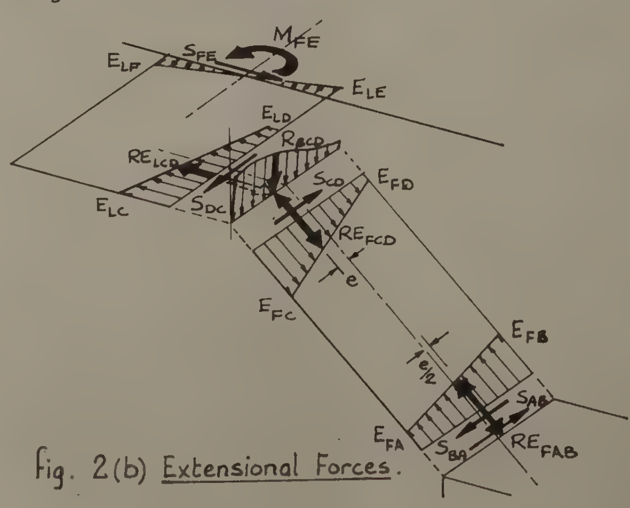

Fig. 2(b) Extensional Forces.

In determining the internal forces the stairs can be considered to be of a homogenous isotropic material, as is common practice in the design of shell roofs and similar structures, and the steel reinforcement thereafter positioned so as to resist all tensile forces in accordance with normal reinforced concrete design practice.

Case II :—

The stairs shown diagrammatically in Fig. 2(a) and consisting of a cantilever landing slab and a single flight rigidly supported at the lower end also complies with the conditions necessary for a complete primary system to act. The primary system which is shown in Fig. 2(b) provides full support to the bending forces at the intersection line CD. The bending forces and local extensional forces are consequently calculated by the normal procedures considering the slab elements to be continuous and to be supported at AB, CD and EF.

As in Case I the resultant reaction $R\beta_{CD}$ at CD will consist of various components depending on the loading and may act at an angle β with the vertical and at an eccentricity 'e' off the centre line of the flight.

If D_{FCD} is the local extensional force in the flight due to the loading on the flight calculated as in Case I then the resultant extensional force in the flight at CD

$$RE'_{FCD} = RE_{FCD} + D_{FCD}$$

where

$$RE_{FCD} = -R_{\beta CD}\frac{\cos\beta}{\sin\theta}$$

and the resultant extensional force in the landing at CD

$$RE_{LCD} = -R_{\beta CD}(\sin\beta + \cos\beta.\cot\theta)$$

As for Case I the force distribution can be assumed to be linear.

Consequently :—

$$E_{FC} = -\left[R_{\beta CD}\frac{\cos\beta}{b\sin\theta}\left(1+\frac{6e}{b}\right)\right] \quad \text{per unit width}$$

and

$$E'_{FC} = -\left[R_{\beta CD}\frac{\cos\beta}{b\sin\theta}\left(1+\frac{6e}{b}\right)\right] + \left[\frac{D_{FCD}}{b}\left(\frac{1+6e'}{b}\right)\right] \quad \text{per unit width}$$

where e' is the eccentricity of D_{FCD}

$$E_{FD} = -\left[R_{\beta CD}\frac{\cos\beta}{b\sin\theta}\left(1-\frac{6e}{b}\right)\right] \quad \text{per unit width}$$

$$E'_{FD} = -\left[R_{\beta CD}\frac{\cos\beta}{b\sin\theta}\left(1-\frac{6e}{b}\right)\right] + \left[\frac{D_{FCD}}{b}\left(1-\frac{6e'}{b}\right)\right] \quad \text{per unit width}$$

$$E_{LC} = +\frac{RE_{LCD}}{b}\left[1+\frac{6e}{b}\right] \quad \text{per unit width}$$

$$E_{LD} = +\frac{RE_{LCD}}{b}\left[1-\frac{6e}{b}\right] \quad \text{per unit width}$$

Considering the equilibrium of the flight

$$RE_{FAB} = RE_{FCD} = -R_{\beta CD}\frac{\cos\beta}{\sin\theta}$$

$$RE'_{FAB} = RE'_{FCD} - W_F\sin\theta = -R_{\beta CD}\frac{\cos\beta}{\sin\theta} + D_{FCD} - W_F\sin\theta$$

(where W_F is the total load on the flight).

$$S_{AB} = S_{CD} = \frac{1}{\sqrt{h^2+l^2}}\left[-RE_{FCD}.\frac{3}{2}e\right] = \frac{1}{\sqrt{h^2+l^2}}\left[R_{\beta CD}\frac{\cos\beta}{\sin\theta}\,\frac{3}{2}e\right]$$

$$S'_{AB} = S'_{CD} = \frac{1}{\sqrt{h^2+l^2}}\left[R_{\beta CD}\frac{\cos\beta}{\sin\theta}.\frac{3}{2}.e - D_{FCD}.e^1 + W_F\sin\theta.e'' + D_{FAB}.e'''\right]$$

where e'' is the eccentricity of the resultant of W_F and e''' is the eccentricity of the local direct force D_{FAB}

$$E_{FA} = +\frac{RE_{FAB}}{b}\left(1-\frac{3e}{b}\right) \quad \text{per unit width}$$

$$E'_{FA} = E_{FA} + \frac{D_{FAB}}{b}\left(1-\frac{6e'''}{b}\right) \quad \text{per unit width}$$

$$E_{FB} = +\frac{RE_{FAB}}{b}\left(1+\frac{3e}{b}\right) \quad \text{per unit width}$$

$$E'_{FB} = E_{FB} + \frac{D_{FAB}}{b}\left(1+\frac{6e'''}{b}\right) \quad \text{per unit width}$$

Considering the equilibrium of the landing :—

$$S_{FE} = -RE_{LCD}$$

$$E_{LF} = \frac{S_{CD}}{a} - \frac{6}{a^2}\left[-RE_{LCD}\left(\frac{b}{2}+e+c\right) + S_{CD}.\frac{a}{2}\right] \quad \text{per unit width}$$

$$E_{LE} = \frac{S_{CD}}{a} + \frac{6}{a^2}\left[-RE_{LCD}\left(\frac{b}{2}+e+c\right) + S_{CD}.\frac{a}{2}\right] \quad \text{per unit width}$$

Case II (b) :—

If the one edge of the flight (BD) were to be built into a wall an additional resistance in the form of a shear force S_{BD} would act along the edge of the flight. Due to the additional support along the edge, the flight could be considered as a slab supported on three edges when determining the bending forces.

Considering the equilibrium of the flight for this case :—

$$RE'_{FAB} = RE'_{FCD} - W_F\sin\theta + S'_{BD}$$

i.e.

$$S'_{BD} - RE'_{FAB} = -RE'_{FCD} + W_F\cdot\sin\theta$$

i.e.

$$S'_{BD} = K_S(-RE'_{FCD} + W_F\sin\theta)$$

and

$$-RE'_{FAB} = (1-K_S)(-RE'_{FCD} + W_F\sin\theta)$$

where K_S cannot be determined by simple methods. A conservative estimate can however be made.

Case III :—

The staircase shown in Fig. 3(a) is of the 'scissors' type being supported at the main landings only. The primary system for this case can never be complete and is statically indeterminate.

The easiest approach in solving this case is to commence by determining the resultants at the intersection lines of the slab elements due to the bending forces and then to draw imaginary straight lines (within the planes of the slab elements) from these resultants to the most rigid supports. The resultant extensional forces will always tend to flow directly towards the most rigid supports. If a set of such direct extensional forces is in complete equilibrium, the primary system may be said to be complete. If, however, equilibrium does not exist, certain types may attain full equilibrium with the assistance of bending forces set up by deformation of

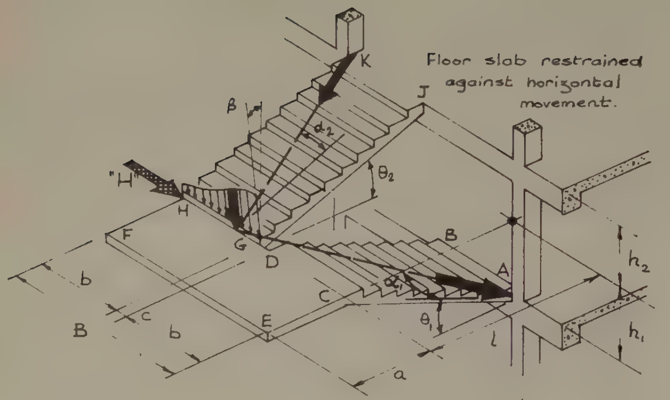

fig. 3(a) CASE III.

fig. 3(b).

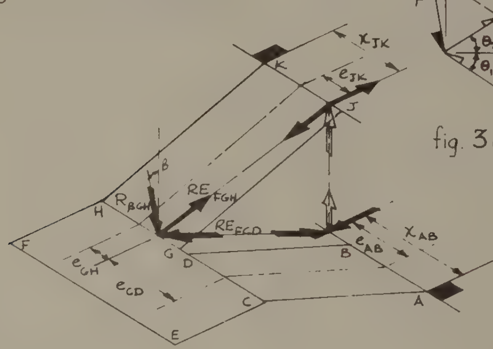

fig. 3(c)

the slab elements. The primary system may then be said to be *incomplete*. The deformation of the structure due to extensional forces is small compared to such bending deformation and may be neglected.

The case under consideration is clearly not in equilibrium under the primary system assumed and would require an additional force 'H' acting as shown and equal to

$$'H' = RE''_{FGH}\sin\alpha_2 - RE''_{FCD}\sin\alpha_1$$

also :—

$$-(RE''_{FGH}\cos\alpha_2\cos\theta_2) - (RE''_{FCD}\cos\alpha_1\cos\theta_1) = R_{\beta GH}\sin\beta$$

$$(RE''_{FGH}\cos\alpha_2\sin\theta_2) - (RE''_{FCD}\cos\alpha_1\sin\theta_1) = R_{\beta GH}\cos\beta$$

(See Fig. 3(b)).

from which 'H' can be determined.

As 'H' is an imaginary force the final solution can now be obtained by applying a force equal in magnitude to 'H' but acting in the opposite direction. This will cause the stairs to undergo a sidesway. This will be resisted by extensional and bending forces in the slab elements causing a displacement of the resultant extensional forces until full equilibrium is attained.

This can only occur when the resultant extensional forces in the two flights act in the same vertical plane so as to have no resultant component in the direction of 'H'. The resultant at A will move towards B and the resultant at K will move towards J thus causing bending in the main landings. Assuming that the deformations due to extensional forces are negligible in comparison with those due to bending of the main landings, the displacement of these resultants X_{AB} and X_{JK} (Fig. 3(c)) will depend primarily on the stiffness of the edges of the main landings in a direction coinciding with the planes of the flights.

If, for example, the upper landing has negligible stiffness compared with the lower landing $X_{JK} = 0$ and $X_{AB} = B$.

If the landings and flights are identical

$$X_{JK} = X_{AB} = \frac{B}{2}$$

and the resultant forces will act at midspan of the main landings. For intermediate cases a precise calculation would be complex but here again an estimate can be made.

The extensional stresses in the flights and landings can now be calculated by a similar procedure as for Case II.

Bending forces will be caused by the loads spanning between the 'supports' as well as by the unbalanced components of the extensional forces and bending shear forces acting at the main landings and along the intersection line CH of the intermediate landing.

Consider the line CH. The unbalanced forces acting on the intermediate landing edge will be as shown in Fig. 3(d).

The unbalanced vertical forces indicated are as a result of the assumption made that the straight line stress distribution applies. The effect hereof is usually negligible but in extreme cases could be dealt with as described previously. The unbalanced forces acting in the plane of the lower flight will be resisted primarily by extensional forces in the intermediate landing and the flights along CD and GH but between D and G considerable bending forces will be induced. The shear forces and bending moments acting in the plane of the lower flight are shown in Figs. 3(e) and 3(f). Bending forces will be induced along the edge DG of the intermediate landing by the vertical components of these forces, whereas the horizontal components will induce extensional forces. In a case such as this where the primary system is incomplete care should be taken as the straight line stress distribution may be erroneous by a considerable amount. As the bending stiffness of the landing is relatively small the force components resisted by bending forces cannot be distributed for the

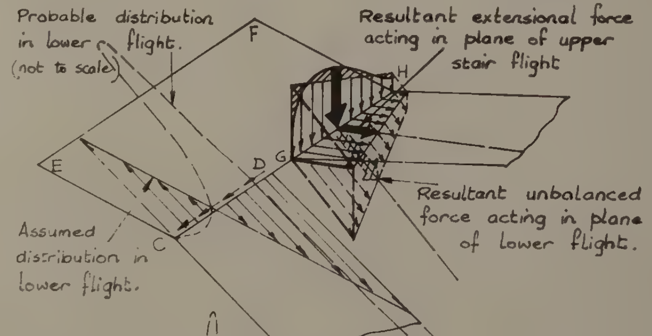

fig. 3(d)(i) EXTENSIONAL FORCES AT CH.

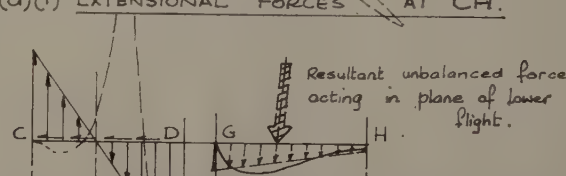

fig. 3(d)(ii) EXT. FORCES IN PLANE OF L. FLGHT.

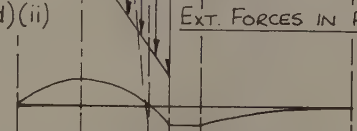

fig. 3(e). SHEAR FORCES IN PLANE OF LOWER FLGHT.

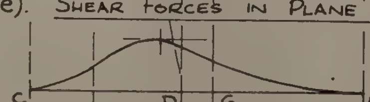

fig. 3(f). BENDING MOMENTS IN PLANE OF L. FLGHT.

full width of the lower flight and consequently a concentration of stress at the inner edge will result. Some allowance should be made for this effect. In most cases in practice the stresses as calculated can be more than doubled without greatly influencing the design. In the case of opposite loading where tensile forces are involved the reinforcement can be concentrated at the inner edge of the upper flight.

Torsional stiffness apart from resisting local forces is not essential for the stability of this type of stairs. In stairs having considerable torsional stiffness a different force distribution will result. Torsion in slab elements has been fully dealt with by various Authors[4]. In stairs of normal dimensions the torsional stiffness of the slab elements is relatively small and can be neglected for the sake of simplicity.

Any loading on the stairs can be treated in a similar manner. For a symmetrical stairs with a uniformly distributed loading the resultant extensional forces (or primary system) will act in a plane on the centre line of the stairs. The unbalanced resultant forces acting at the midspans of the main landings will be resisted by bending forces in combination with secondary extensional forces. Although the stair flights and intermediate landing slab can be relatively thin the main landings will have to be capable of supporting in effect the total weight of the stairs at midspan.

The extensional forces acting at the lines of intersection of the slab elements in a stairway of this type with a uniformly applied distributed load is shown in Fig. 3 (g) (i).

β_1 and β_2 will in most cases be small and the reactions at the intersection lines can for simplicity be considered to act vertically.

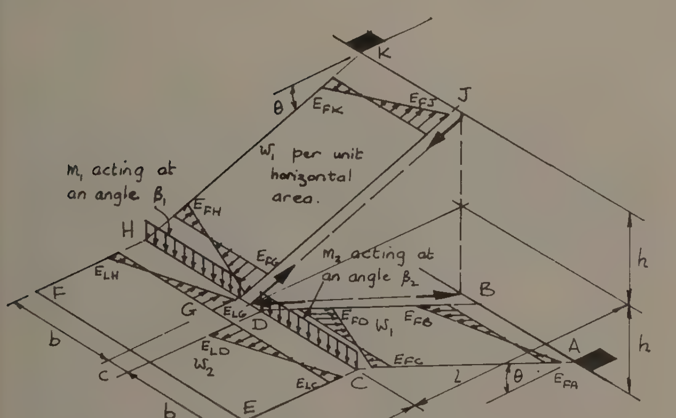

fig. 3(g)(i)

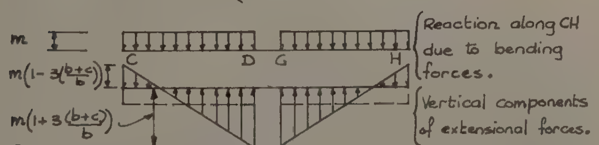

fig. 3(h)(i) FORCES ACTING IN VERTICAL PLANE.

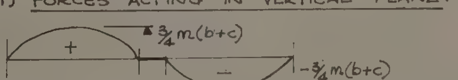

fig. 3(h)(ii) SHEAR FORCES ACTING IN VERTICAL PLANE

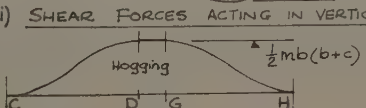

fig. 3(h)(iii) BENDING FORCES ACTING IN VERT. PLANE.

$$m_1 = m_2 = \left[k_1 w_1 lb + k_2 w_2 a\left(b + \frac{c}{2}\right)\right]\frac{1}{b} = m$$

$$RE_{FCD} = -RE_{FGH} = -\frac{mb}{sin\theta}$$

$$RE^1_{FCD} = -RE^1_{FGH} = -\frac{mb}{sin\theta} + \frac{w_1\, lb\, sin\theta}{2}$$

The magnitudes of the extensional forces are as follows :—

$$E_{FA} = +\frac{RE_{FCD}}{b}\left[1 - \frac{3\,(b+c)}{b}\right]$$

$$= -\frac{m}{sin\theta}\left[1 - \frac{3\,(b+c)}{b}\right] \quad \text{per unit width}$$

$$E'_{FA} = +E_{FA} - w_1 l\,\frac{sin\theta}{2} \quad \text{per unit width}$$

$$E_{FB} = +\frac{RE_{FCD}}{b}\left[1 + \frac{3\,(b+c)}{b}\right]$$

$$= -\frac{m}{sin\theta}\left[1 + \frac{3\,(b+c)}{b}\right] \quad \text{per unit width}$$

$$E'_{FB} = +E_{FB} - w_1 l\,\frac{sin\theta}{2} \quad \text{per unit width}$$

$$E_{FC} = E_{FA} \text{ and } E'_{FC} = E_{FC} + \frac{w_1 l\, sin\theta}{2} \quad \text{per unit width}$$

$$E_{FD} = E_{FB} \text{ and } E'_{FD} = E_{FD} + \frac{w_1 l\, sin\theta}{2} \quad \text{per unit width}$$

$$E_{LC} = -\frac{m}{tan\theta}\left[1 - \frac{3(b+c)}{b}\right] \quad \text{per unit width}$$

$$E_{LD} = -\frac{m}{tan\theta}\left[1 + \frac{3(b+c)}{b}\right] \quad \text{per unit width}$$

$$E_{LG} = -E_{LD} \text{ and } E_{LH} = -E_{LC} \quad \text{per unit width}$$

$$E_{FG} = -E_{FD} \text{ and } E_{FH} = -E_{FC} \quad \text{per unit width}$$

$$E'_{FG} = E_{FG} - \frac{w_1 l\, sin\theta}{2} \text{ and } E'_{FH} = E_{FH} - \frac{w_1 l\, sin\theta}{2}$$

per unit width

$$E_{FJ} = -E_{FB} \text{ and } E'_{FK} = -E_{FA} \quad \text{per unit width}$$

$$E'_{FJ} = E_{FJ} + \frac{w_1 l\, sin\theta}{2} \text{ and } E_{FK} = E'_{FK} + \frac{w_1 l\, sin\theta}{2}$$

per unit width

The unbalanced resultants of the primary extensional forces along CH act in this case in a vertical plane and are shown in Fig. 3 (h) (i).

The shear forces and bending moments due to these forces are shown in Fig. 3 (h) (ii) and (iii).

By similar methods the bending and extensional forces at any section can be determined.

Note that the assumption has been made that the vertical reaction of the bending shear forces at CH is uniformly distributed. This is a conservative assumption as the load concentration will actually be higher towards D and G, thereby causing smaller bending moments on CH.

Case IV :—

The staircase shown in Fig. 4 is similar to Case III with the exception that the end of the intermediate landing is restrained horizontally and vertically as shown. The extensional forces for any loading can be determined by a similar procedure. If the wall supporting the landing is relatively 'rigid' in comparison with the vertical bending stiffness of the main landings the moments caused by sidesway will be resisted within the flights and intermediate landing by extensional

and bending forces, the result being a 'shift' of the resultant extensional forces as shown.

$R_{\beta CD}$ is determined from the bending forces.

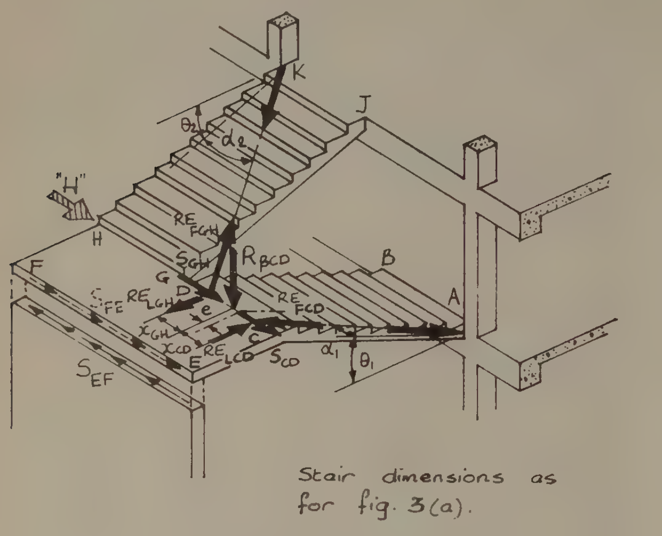

Fig. 4

Considering the equilibrium of the intermediate landing :—

$$R_{\beta CD}.\ \cos\beta = - RE_{FCD}.\ \cos\alpha_1\ \sin\theta_1 + RE_{FGH}.\ \cos\alpha_2.\ \sin\theta_2 \qquad (1)$$

$$RE_{LGH} + R_{\beta CD}.\ \sin\beta = - RE_{LCD} \qquad (2)$$

where

$$RE_{LGH} = RE_{FGH}.\ \cos\alpha_2.\ \cos\theta_2 \qquad (3)$$

$$RE_{LCD} = RE_{FCD}.\ \cos\alpha_1.\ \cos\theta_1 \qquad (4)$$

$$S_{FE} = S_{GH} + S_{CD} \qquad (5)$$

where

$$S_{GH} = RE_{FGH}.\ \sin\alpha_2 \qquad (6)$$

$$S_{CD} = - RE_{FCD}.\ \sin\alpha_1 \qquad (7)$$

$$RE_{LGH}.\ X_{GH} - RE_{LCD}.\ X_{CD} = S_{FE}.\ a \qquad (8)$$

$$RE_{FGH}\ \cos\alpha_2.\ \sin\theta_2.\ X_{GH} + RE_{FCD}.\ \cos\alpha_1 \sin\theta_1.\ X_{CD} = 0 \qquad (9)$$

$$\frac{3}{2} b + c - e - X_{GH} = \frac{l}{\cos\theta_2}.\ \tan\alpha_2 \qquad (10)$$

$$\frac{b}{2} + e - X_{CD} = \frac{l}{\cos\theta_1}.\ \tan\alpha_1. \qquad (11)$$

The internal extensional forces can consequently be determined.

The secondary extensional and bending forces can be determined as before.

If the wall supporting the intermediate landing is not 'rigid' the solution will be somewhere between that of Cases III and IV.

Case V :—

If the intermediate landing is also restrained at the two sides CE and FH as shown in Fig. 5, complete equilibrium can be attained by the set of extensional forces shown which can be determined as before by simple equilibrium equations.

If the wall restraints are not 'rigid' the solution will be somewhere between that of Cases III and V, depending on the relative stiffnesses.

Note that loads applied to the main landings will also be supported by the extensional force system in this particular case as the points C and H are restrained against deflection by the wall supports.

Complex Primary Systems :—

In stairs having one or more redundant restraints two or more primary systems may operate conjointly. It is usually possible to make a reasonable estimate of the distribution of the load between such systems based on the estimated relative stiffnesses. As the stresses are usually small it is not necessary to achieve great accuracy therein. It is furthermore suggested that the following propositions may be an additional justification for this approach to the problem.

1. *If any slab type stairs or similar structure in which the slab thickness is small compared with the overall dimensions is subjected to an applied load or set of loads any stable primary system or set of systems, as defined previously, may be selected and provided that the sum of the stresses induced in the component parts by the primary forces and the secondary bending forces for such a system are not in excess of ultimate stresses or do not cause instability, then such a primary system or some other primary system, able to sustain an equal or greater loading, will operate effectively before failure occurs.*

This proposition is analogous to the load factor method used in the design of beams and frames. It is based on the assumption that if for any particular loading the actual stresses at a point in the structure exceed that given by the analysis based on the assumed system, then before the ultimate load resistance is attained the systems will alter so as to accommodate the forces acting, much in the same way that moment redistribution occurs in frames.

2. *The correct solution is that system or set of systems for which the total internal strain energy is a minimum. The total strain energy will be a minimum when the strain energy due to bending forces is a minimum or in other words the correct solution is that which, provided that stability is achieved, causes a minimum of bending in the slab elements.*

This proposition is precisely true only for a structure in which the thickness of the slab elements is very small compared with the planar dimensions. It will, however, be approximately true for most cases occurring in practice.

The application of the Method in practice :— It must be emphasized that this method of analysis cannot be applied indiscriminately to all types of structures as the errors inherent in the assumptions made may be significant in certain cases. For stairs of the normal dimensions dictated by practical considerations the extensional stresses are, however, small compared with the bending stresses, so that even if a considerable error

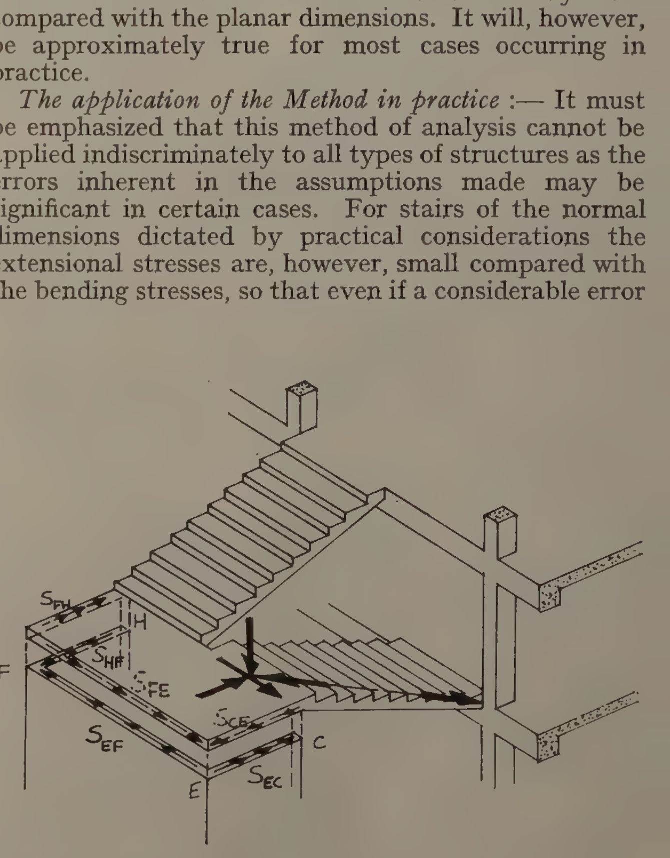

Fig. 5

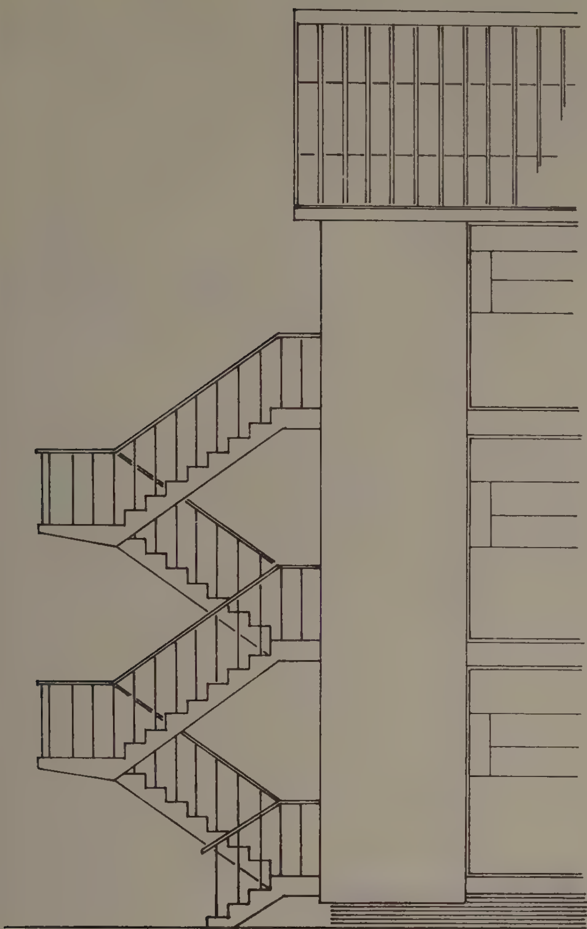

Fig. 6

Fig. 7

is made it will not necessarily affect the value of the total stresses by very much. The analysis of the extensional stresses within the slab elements after the determination of the stresses or forces at the boundaries (intersection lines or supports) can be based on the same assumptions.

In actual fact the problem is not unlike the deep girder problem, the solution of which is difficult. In certain obvious cases much can be accomplished by intuitive ' scientific guessing ' based on known results.

It is essential that the external supports to the internal extensional forces should be sufficiently rigid as a small horizontal displacement may cause considerable vertical deflection of the ' Supports ' at the lines of intersection of the slab elements depending on the proportions of the stairs.

Where the external supports are relatively rigid (i.e. floor slabs with many restraints) the deflection of the ' supports ' are, however, negligible. In some cases where the external restraints are not rigid or where doubt as to the efficacy of these restraints may exist, it may be possible to make an estimate of the forces acting and an approximate determination of the resultant deflections. The effect of the deflection of the ' supports ' provided by the primary system may then be allowed for in the determination of the bending forces. In this determination great accuracy is not required if we bear in mind the latest developments of the plastic and ultimate-load or load-factor methods of design in which a considerable re-distribution of bending moments is allowed. It is no less difficult to make an accurate determination of actual bending moments in floor slabs supported on beams especially where the beams are relatively light and subject to considerable deflection.

Although tests have indicated that the restraint provided by brickwork can be considerable, it is to a large extent an unknown factor bearing in mind the possibility of a shrinkage 'gap ' between concrete and brickwork and the variable properties of brickwork. Precautions should be taken to ensure positive support where brickwork is used to provide external restraint. Where the extensional forces are small in magnitude frictional forces may, however, provide sufficient resistance.

The fire escape stairs illustrated in Fig. 6 have been designed for a block of flats to be constructed near Cape Town and the photograph (Fig. 7) shows another type in a building in Oudtshoorn, which is typical of several stairways constructed with complete success. The brick wall to the right is a boundary wall and the intermediate landing is not cantilevered from it, but is supported by the flights.

Notation

$R\beta_{CD}$ —is the resultant reaction due to the secondary bending force system and local direct forces acting at the intersection line CD at an angle β with the vertical.

R_{FCD} —is the reaction at CD due to the bending forces in the flight acting at right angles to the flight.

R_{LCD} —is the reaction at CD due to the bending forces in the landing acting at right angles to the landing.

D_{FCD} —is the reaction at CD due to the local direct forces in the flight acting in the plane of the flight.

RE_{FCD} —is the resultant extensional force in the flight at CD due to the reaction $R\beta_{CD}$ but not including the effect of the local direct forces in the flight.

RE'_{FCD} —is the resultant extensional force in the flight at CD due to the reaction $R\beta_{CD}$ and including the effect of the local direct forces in the flight.

RE_{LCD} —is the resultant extensional force in the landing at CD.

E_{FC} —is the extensional force per unit length in the flight at C not including the local direct force.

E'_{FC} —is the extensional force per unit length in the flight at C including the local direct force.

E_{LC} —is the extensional force per unit length in the landing at C.

S_{CD} —is the shear force acting along the intersection line CD due to the primary force system but not including the effect of local direct forces.

S'_{CD} —is the shear force acting along the intersection line CD due to the primary force system and including the effect of local direct forces.

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Andrews, W. C., The Elliot Secondary School, Putney. *The Structural Engineer*, Nov., 1956.

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Discussion

The Council would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by 30th July 1960.

Book Reviews

Handbook of Heavy Construction, edited by F. W. Stubbs. (New York and London : McGraw-Hill, 1959). 9 in. × 6 in., 1040 pp., £7 3s. 6d.

This American handbook gives the methods, data and working information required in all branches of heavy construction. The contributing authors are specialists in their respective fields, each with considerable experience, and collectively they represent active contractors, manufacturers, distributors and consulting engineers.

The book is divided into nine sections, the first dealing with excavation and transportation of earth and rock, and including a section on dewatering. The second section on concrete has chapters on selection of materials and mix proportions, aggregates, concrete batching, mixing, placing and curing, pneumatically applied concrete, instrusion grouting, precast and prestressed concrete, reinforcing steel and formwork.

Section 3, dealing with steel, has chapters on steel erection, electric arc-welding and flame cutting and welding, and the following three sections are on heavy timber construction, bituminous pavements and cross country pipelines. Section 7 on foundations deals with piles and pile driving, cofferdams and caissons, and the following section on miscellaneous equipment and operations, includes floating equipment and river diversion. The volume concludes with chapters on contractor's organization and planning, the work of a resident engineer, construction contracts, and planning for safety.

In addition to being a comprehensive work of reference for contractors, this handbook will also be of value to consulting engineers, designers, architects and to students wishing to obtain information on the subjects covered. The volume is well produced and contains more than six hundred illustrations.

Analysis of Continuous Beam Bridges, Vol. 1, Carry-Over Procedure, by J. J. Tuma, S. E. French and T. I. Lassley (Oklahoma State University School of Civil Engineering Research Publication No. 3, 1959). 11 in. × $8\frac{1}{2}$ in.

This report, which is the first publication in a series entitled " Analysis of Continuous Beam Bridges," deals with the application of the numerical carry-over moment method to the analysis of continuous beam bridges subjected to stationary or moving loads, applied couples, displacements of supports or change in temperature. The method is a numerical successive approximation, which may be carried out to a desired degree of accuracy. The study is restricted to coplanar systems and the customary assumptions of beam analysis are introduced.

The publication is divided into six parts. The derivation of the general three moment equation for beams of variable section and the definitions of the beam constants are given in the first chapter. The geometry and the integral functions of beams with parabolic haunches are discussed in the second chapter. The algebraic formulae of beam constants such as angular flexibilities, carry-over values and load functions for beams mentioned before are derived in the third and fourth chapters. The numerical evaluation of these algebraic formulae is made by means of a high speed computer and the results recorded in tabular form in the fifth chapter. The application of the numerical carry-over procedure in connection with the tables of beam constants is illustrated by two examples in the last chapter.

The Design and Construction of the New Basic-Bessemer Plant at Port Talbot *

Discussion on the Paper by J. W. P. Jaffé, M.A., (Cantab.), A.M.I.Struct.E., A.M.I.C.E.

THE PRESIDENT introduced the Author, who then gave a precis of his paper, illustrating it with a series of diagrams and photographs on the screen.

The PRESIDENT, complimenting Mr. Jaffé on his paper, which was extremely well set out and clear, expressed the thanks of the Institution to him for having presented it, for the exposition of it he had given at the meeting and for the very good pictures he had shown to illustrate the structure described.

It might well be borne in mind by any of the audience who contemplated giving papers to the Institution in the future that it was a good thing to make slides in colour, for they added a good deal to the presentation of Mr. Jaffé's paper; they were very good pictures and they showed the cleanness of the structure in general.

Discussion

Mr. S. M. REISSER (Member) called attention to the author's statement on p. 289, under the heading "High Strength Pre-Stressed Bolts," that these bolts were used in all connections subject to tensile and fatigue loading, where welding was unsuitable. He asked why welding could not be used in the portal caps.

Mr. R. F. GODMAN (Member) asked if the author had considered increasing the stress in the grip bolts beyond the minimum specified yield stress. Another question was that where a connection could not be completely made at once if, for instance, a period of a few weeks or even a few months elapsed before the bolts were tightened up, did he find any trouble with rust formation lowering the value of μ. The value specified for μ of 0·4 was quite a high one.

Also were the bolts parallel shank or waisted? And how was specified 'turn of nut' measured?

Mr. B. L. CLARK (Member) commented that Mr. Jaffé seemed to have implied that structural engineers did not know much about steelworks and the requirements of steel works processes, and said he did not know if this was so. Some of them knew quite a good deal about cranes and about the plant in steelworks. He believed that failures which had occurred had been mechanical ones, principally on cranes where the client had not had his requirements sufficiently known, or where attempts had been made to lighten them beyond the limit which time had proved expedient.

There had been many wheel troubles for which the crane makers were to some extent responsible since they had not appreciated that a steel structure was elastic and there must be considerable margins of reserve strength and clearances.

Frequently one heard reference to making lighter cranes by using higher strength materials. Mr. Clark, however, urged it was dangerous, and in the long run uneconomic, to make cranes light simply on the basis of stress requirements, because in many instances the load lifted could overpower the girder or crane as a whole.

It was a most important feature in any structure in which machinery was used—and especially cranes where demands for high speed and positive control were experienced—that there must be ample inertia at self weight in all the motions.

Mr. GODMAN having seen steelwork cranes and the terrific usage of them agreed with Mr. Clark that there should be no attempt to produce light structures for these.

Mr. L. R. KNOTT (Associate-Member) asked if the Author would elaborate on his statement on p. 286, in the paragraph immediately above the heading 'Crane Girders,' that the bending moment analysis was carried out in a conventional manner and checked by means of a computer. He also asked whether anything interesting came out of that.

Mr. F. R. BULLEN (Vice-President), referring to the author's special detail of rail fixing (Fig. 7), asked whether the crane rail arrangements described in the paper had proved satisfactory in use.

He was also interested in the surge linkage (Fig. 4) and asked for some more information about it. It seemed to him quite a new idea, although his firm had in fact used something similar, but not the same in detail. He did not quite follow the argument in the paper, not because it was not well put, but because he had not studied it in detail.

Mr. Bullen had some sympathy with the view expressed by Mr. Clark and Mr. Godman that we must not reduce too far the weights of steel used. It was not just a question of reducing the weight of steel, but of reducing cost, and that was not always the same thing as reducing the weight of steel. Mr. Jaffé had given some figures, and if he could give any in terms of (say) lb. per cu. ft. it would be helpful.

Endorsing the President's remarks about the quality of the paper and the slides by means of which it was illustrated, he said that, whilst he would have liked to have seen a few more details, because they were always useful, particularly to students, as well as to older members of the Institution, he must agree that the paper was well produced and the diagrams were well worth studying. As the President had indicated, it might well serve as a model for other papers.

Finally, in a reference to the layout plan in Fig. 1, he said there were figures from 1 to 13 in single circles and others up to 18 and the letters TF, TE, and so on, in double circles. He asked if those symbols could be explained.

Mr. J. HINTON (Associate-Member) recalled that he had been concerned with Messrs. Simon-Carves at Abbey Works on the coke-oven foundations and said they had run into trouble due to settlement of the heavy slag fill carrying some of the piles down; there was some 15 or 20 ft. of fill. He asked if anything like that had occurred at the new works described in the paper.

Mr. L. E. WARD (Member), inviting Mr. Jaffé to say a little more about the roof plates, said that the plate thickness of $\frac{7}{32}$ inch seemed to be rather thick for a spacing of 3 ft. $1\frac{1}{2}$ in.

Mr. B. C. BIRD (Associate-Member) observed that it was not quite clear from the cross section what type of bases were used under the building stanchions. They

*Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1., on the 22nd October, 1959. Mr. L. E. Kent, B.Sc., M.I.Struct.E., M.I.C.E. (President) in the Chair. Published in "The Structural Engineer," Vol. XXXVII, No. 10, pp. 282-291 (Oct. 1959).

were apparently the usual Abbey dustbin foundations, and Mr. Bird wondered whether Mr. Jaffé would care to confirm this.

Mr. O. Bondy (Member) said he was reluctant to ask questions because he had not had time to give the paper the amount of study it certainly deserved. But remarks by earlier speakers led him to ask a supplementary question regarding Fig. 3. It had been asked whether it was necessary to make the roof plates $\frac{7}{32}$ inch thick. His own query was, if that thickness were necessary, why a $\frac{3}{16}$ inch leg size used for the continuous sealing weld along the upper layer of plates? Were there any stresses transmitted, perhaps due to wind suction?

Under the heading 'Welding' on page 289 there was reference to the use of electrodes having a Class No. 6 covering to B.S.1719. He asked if he were right in thinking that these electrodes were of the low hydrogen type.

Concerning the testing of butt welds in the crane girder flanges, he said that ultrasonic examination was specifically mentioned, and he asked if that followed the regular radiographic examination.

Mr. B. L. Clark asked whether any observations had been made on the crane rails since they were put into service, and whether rubber pads under the rails had proved to be of benefit to the crane over a long term.

It was undeniable that the rubber pads considerably reduced impact, but here again if crane girders had excess of material in self weight, time had proved that they were more serviceable and had a greater margin to absorb impact.

He would like to know how the rubber behaved, whether in fact fretting had been experienced and how far it had helped the efficient operation of the crane.

The President pointed to the statement, in item seven on page 283, which referred to an office block of reinforced concrete construction sited west of the mixer bay. That seemed a little interesting to him.

Another statement to which he drew attention, in the second paragraph in the second column of the same page, was: " . . . it was decided to adhere throughout to the stanchion spacing of 50 ft. dictated by the plant. This dimension also represents the longest length of crane girder which can be fabricated without introducing splices in the flange and web plates." The President thought there was an Abbey Works tradition in this particular matter and, if they did use splices in the flanges of the crane girder, perhaps that tradition was not carried out in this particular building.

From the fourth paragraph in the same column he noted that only one expansion joint in the crane girders, on line 10, was provided in the length of the building, which gave something of the order of 300 ft. plus on either side of the central line of the building. He wondered if Mr. Jaffé had a note of any movement in either of those 300 ft. lengths

The President joined Mr. Bullen in asking for additional information concerning the surge linkage, illustrated in Fig. 4.

Lastly, he said he did not understand the statement, in the last paragraph of page 286, that: "It was found that to join the adjacent girders together rigidly with surge bracing would lead to impossibly large forces in the bracing members when only one girder was loaded." He asked for more information about that statement.

Mr. Jaffe replied to the discussion.

Dealing first with Mr. Reisser's question as to where high strength pre-stressed bolts were preferred to welding, he said a lot of these bolts were used in connecting the telpher runway hangers to the supporting structure. Slotted holes had been provided so that the running rail could be lined and levelled after erection. The firm who supplied the telpher cars had suggested that the slotted holes be finally welded up, but it appeared better to make use of high strength bolts to prevent movement. They were also used in places where metal spray would have been destroyed by welding.

Replying to Mr. Godman he said that of the high strength bolts used in the basic-Bessemer plant the long crane girder holding-down bolts were waisted and the short ones in the telpher supporting structure were not. They were all used under cover and, probably because of this, no cases of rusting had been noticed between the time of erection, when black bolts were used, and the time when the high-strength bolts were inserted and pre-stressed.

The method of tightening was designed to ensure that the bolts yielded and, so far as he knew, that always took place.

Concerning the weights of cranes, referred to by Mr. Clark, Mr. Jaffe's point was that the crane maker was well aware of fatigue life, but his only counter was to reduce the working stresses by putting up the weight of the crane. He was forced to do so because he could not, under the usual system of tendering, afford to quote a price based on the use of notch ductile steels. Consequently he tended to put forward a crane which was unnecessarily heavy. The trouble was that the designer of a building and the crane maker never got together and looked at the job as a whole. It was neither the cheapest building nor the cheapest crane that was wanted, but the cheapest combination of building and crane, and a lot remained to be done to bring this about.

As to the running qualities of the crane on the rubber mounted crane rail, experience at the Steel Company of Wales' plants had been very good. Although the Bessemer plant had not been operated for long enough to be conclusive, the same method of rail fixing at the cold reduction plant at Velindre, which had been in commission for a number of years, had given excellent results and there was no reason to modify anything that had been used there. The cranes were almost noiseless in operation.

The question was raised by Mr. Knott as to whether anything interesting came out of the computer analysis. Mr. Jaffé supposed that the answer was no, as the computer confirmed their design to within 3 per cent.

He went on to say that the computer was not in this instance used to the best advantage, for they had used it only as a means of checking the bending moment analysis carried out by moment distribution. The computer was, of course, a powerful tool and could be used to refine designs in a very short space of time. For a complicated design such as that at Port Talbot, it would take about a fortnight to get an answer by conventional methods and it was not usually possible to carry out more than two designs before sections had to be ordered from the mills. With the computer however, one could afford to check a number of designs and eventually arrive at the most economical solution.

Replying to the request by Mr. Bullen and the President for more information concerning the surge linkages, he said the problem was that if two adjacent crane girders were braced together rigidly in a horizontal plane and one carried a vertically applied load, the fixed end moments imposed on the bracing members by the differential deflection would be very high indeed. Such a system of bracing was subject to fatigue loading and could not be employed with any degree of success. It had therefore been decided to use the linkage

shown in Fig. 4 in which shearing of the natural rubber insert enabled the outer tube to rotate in relation to the inner tube. This meant that one girder could deflect vertically in relation to the other without thereby stressing the linkage. In the basic-Bessemer plant the converter bay crane girder with its outrigger girder provided torsional and surge stiffness to the mixer bay crane girder. The linkage was a novel feature which the designers have introduced as a result of a series of trials.

Dealing with Mr. Kent's question as to whether there had been any settlement of the plant, he said as far as he was aware there was none. That might be due to the fact that there was virtually no floor loading in the greater part of the works. The only heavily loaded floor was that in the dolomite bay and that was piled. No settlement had been reported.

The foundations followed the practice established at Abbey Works of solidly casting the stanchions into sockets. This presupposed that foundation movement would be of such small order that any adjustment that became necessary could be taken up at crane girder level. Some of the crane girders at Abbey Works had been in commission for upwards of ten years and he did not think there was a case, save in the open gantry slab yard, where the floor loading was exceptionally heavy, in which foundation movement had given rise to trouble.

Mr. Jaffé was disappointed that the meeting had not raised the question of why the stanchion which supported the converter trunnion was wedge-shaped. The reason was that almost immediately below its foot there was a services tunnel and it had therefore been impossible to provide any end-fixity to this stanchion. They had had to make it pin ended and expressed this feature in the shape of the stanchion.

The design of the roof plates was somewhat complicated by the fact that the Steel Company of Wales had talked of having a mechanical dust sweeper. He did not know quite what they had in mind, and he was not too sure that they did, and so the designers had played safe. They had assumed that a kind of mechanized lawn mower would sweep up the dust and designed the plate for a heavy point load.

As to the stresses likely to occur on the roof, mentioned by Mr. Bondy, he said it was perhaps not obvious from the illustrations, but there was only sufficient angle bracing in the plane of the roof to square the building during erection. The roof plate was designed to take wind and expansion stresses, and the welds had to be adequate to transmit these stresses. The roof became very hot in summer and cold in winter and they wanted to make sure that repeated temperature movement did not cause cracking of the welds. The welds were also required to seal the plates and to prevent corrosion where the high plates lapped over the low plates.

Coming to the President's remarks, Mr. Jaffé explained that the reason why he had made a point of saying that the office block was of reinforced concrete construction was that it was almost 'treasonable' to put up this sort of structure on a steelworks site. But reinforced concrete construction was used, because steel could not have been obtained in time to meet the programme for this building.

With regard to the point about splices in the crane girders he should have said that the 50 ft. girders which were fabricated in the shops could be made from single flange and web plates. On site the girders were welded up continuously, and there they had full strength butt welds.

As to temperature movement, they had made use of the fact that the crane girders were high above the shop floor. As a result the temperature stresses at the base of the columns were low. There must have been quite a lot of expansion movement last summer, but they had provided an adequate expansion gap on line 10 and, so far as he knew, it had proved satisfactory.

Referring to Fig. 1, he explained the purpose of the single and double lines around the figures. Endorsing the view expressed during the discussion that the diagrams in the paper were very clear, he said that he felt able to do so because when they were prepared he was in Canada. He owed a debt of gratitude to his colleagues for adding the finishing touches to the paper when he was some 3,000 miles away.

THE PRESIDENT, at the conclusion of the discussion, commented that for many years structural engineers had been seekers after quality and not necessarily quantity, and there was quality in the questions which had been put at the meeting; there was quality, too, in the manner in which Mr. Jaffé had replied to them. He had evaded none of the questions and had given forthright answers to them all. The meeting's thanks were due to him for the obviously able way in which he had done so.

Corrigendum

"*The Structural Engineer,*" p. 290, col. 1, line 3. *For* 2 per cent proof stress *read* 0·2 per cent proof stress.

Book Review

A Testament, by Frank Lloyd Wright. (New York: Horizon Press. London: Architectural Press, 1957). 12 in. × 9½ in., 256 pp. 70s.

Divided into two books—"Autobiographical" and "The new architecture"—here is the outpouring of an architectural genius. His father a minister who loved and taught music, his mother a teacher who loved teaching, he undoubtedly inherited from them his love of learning, his love of art and his love of truth. Victor Hugo's "Notre Dame" read when he was a boy, also had a deep influence on his architectural development.

His first post with Adler and Sullivan was a propitious one, and his long career creating organic architecture began. Whilst being thrilled with Mayan, Inca and Egyptian remains and loving the Byzantine, he felt he owed no debt to classical architecture, and the beaux-arts of Paris had no influence upon him. The Chicago World Fair of 1893 filled him with dismay, because it set back organic architecture and gave a new lease of life to the classic.

Reading through the book and studying the superb drawings and photographs of his works, one is awed by his single mindedness, his profound thinking and the prodigious amount of work he carried out. He was not diffident about his achievements, but was ever ready to tell how he pioneered. Looking again at the photograph of Kaufmann House, "Fallingwater," Pennsylvania (1936) he tells us that this was the first house in his experience to be built of reinforced concrete—"so the form took the grammar of that type of construction." And how well this building fits in with the rocks and surrounding landscape.

In book two we are given a synthesis of the nine guiding principles upon which his life's work was based—this makes most interesting and illuminating reading indeed.

A great book by a great master. G.D.

Multi-Storey Car Parks*

Discussion on the Paper by E. N. Underwood, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., (Vice-President)

THE PRESIDENT proposing a vote of thanks to the Author, congratulated Mr. Underwood on his up to date paper and said the subject of off-street parking was very much in our minds and would be of considerably more importance in the future than it had been in the past.

Congratulations were also due to Mr. Underwood for having made his analysis in the early part of the paper and for having kept strictly to the type which he had found or had estimated to be the most economical type; then he had continued without allowing any other consideration to dissuade him from his purpose.

MR. UNDERWOOD, responding, emphasised that the scheme was devised when he had a Partner, Mr. Mander, who was present at the meeting and had been closely associated with the work from its inception. Mr. Mander should feel, therefore, that he also shared in the reward.

MR. DONOVAN H. LEE (Member of Council) said, the paper was so well presented that there might be no questions left to ask. However, although it all seemed so logical it was interesting to consider what alternatives there had been and to learn the Author's views on them.

There was a similar garage in Sydney with a very low storey height. It seemed to him that, since we must have a 'strong point' to give the lateral stability, there was little economy in restricting the storey height. As regards the 'strong point,' in a garage of this kind in San Francisco this was done in prestressed concrete with a large number of high tensile bars but was partly to provide resistance to earthquakes.

He asked if the Author had considered taking advantage of prestressing in the circular part of the structure with the big cantilevers. In Johannesburg a garage was now being built with big spans of 50 or 60 feet and the beams were prestressed. Mr. Lee asked for the Author's views on the possible use of both precasting and prestressed concrete in future similar cases.

MR. UNDERWOOD replied that in the paper he had dealt with storey height. The view of Mr. Mander and himself was that if, for example, a height of 7 ft. 6 in. were provided, which he understood was the case in certain parks, car owners would use the garage while there was no alternative, but immediately somebody opened another garage with more headroom they would go to that one. With a 56 ft. width of deck, a man driving up to the centre would feel very depressed if the roof or beam were just above his head; the park had been designed therefore on a 10 ft. deck to deck height, which after all only meant 1 foot or so on the columns at each circuit.

If they were ever faced, and they hoped they would be, with the "double thread worm" type of structure, and wished to preserve the 10 ft. deck to deck dimension, they would have to face a climb of 20 ft. per circuit, whilst still preserving a gradient of about one in thirty. In order to do that, a 600 ft. length of road was needed, and sites would be extremely costly if this 600 ft. perimeter could be achieved. So that immediately the possibility of having more than (say) 500 cars in the park arose, it became necessary to consider double thread worms and to think also in terms of 9 ft. or 9 ft. 6 in. deck to deck. This meant a climb of 18 ft. or 19 ft. per circuit.

The loading was 80 lb/sq. ft. on the deck and 50lb/sq. ft. on the beams, and that allowed for cars up to 2½ tons. To cater for heavier vehicles would be unwise. They could be accommodated on the ground floor, but again one would be up against the problem of headroom, because some vans were very tall. That trouble had already arisen in connection with a petrol filling station on one side of a park, and one had to face up to it.

A SPEAKER asked whether, instead of catering only for cars, the structure could be a multi-purpose garage, for surely that would be ideal.

MR. UNDERWOOD replied that that suggestion had been considered, but he thought the siting of the garage precluded any chance of getting coaches into it. They were better kept on the ground, for they were heavy; it would not be economical to provide a structure in which they could be mixed with cars. If the minicars were turned on to the roads in the numbers predicted, it might well be possible to cater for them up to 12 ft. long, to provide special accommodation for them with a smaller width than for normal cars, and less headroom, thereby achieving terrific economy. We could not establish a park in a business and shopping area and limit it to the accommodation of that small type of car however. For very long cars there could be bays of 20 ft. length on one side of the deck, and bays on the other side could be reduced in length from 16 ft. to 12 ft. for use by mini-cars. This would only involve painting the white lines in different positions. The average car on the British roads would go into a 16 ft. bay easily and the access road, 24 ft. wide, would provide sufficient room for passing, manoeuvring and so on.

MR. K. H. ONG (Associate-Member), pointing to Fig. 1, asked if the fishbone type of parking would not enable one to park a car more quickly than would the other types. He was not sure which was the more important, to accommodate the maximum number of cars in a given area or to provide for quick and easy parking.

MR. UNDERWOOD replied that the matter was dealt with on the basis of areas. One had a parking bay 16 ft. by 8 ft. placed alongside a stretch of road 8 ft. by 12 ft.; so that the area of the car bay was 57 per cent of the total area of the 8 ft. strip measured to the centre of the access road. (62 per cent with a 20 ft. road). When parking on the fishbone pattern the car had to reverse out of the bay and then go forward in the direction in which it had entered; so that the fishbone type of parking was tied really to the "one-way" type of road access.

**Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1., on the 10th December, 1959. Mr. L. E. Kent, B.Sc., M.I.Struct.E., M.I.C.E. (President) in the Chair. Published in "The Structural Engineer," Vol. XXXVII, No. 12, pp. 349-356. (Dec. 1959).*

Various systems had been shown in diagrams which were one-way only and, having arrived at a peak, one turned a circle and started going down hill, on another road system within the garage. Using that type of structure, the fishbone type of parking was, of course, very attractive ; but for operation, where cars entered and left on the same road, certainly parking normal to the road was the proper thing.

Mr. D. R. Sharp (Associate-Member of Council) was very interested to read in the paper some of Mr. Underwood's ideas on car parks and to compare them with his own observations of parking garages in America.

He considered that Mr. Underwood had been very wise to increase the internal access roadway width from 20 feet to 24 feet because in America one gained the impression that many access roadways were not wide enough. As a result some drivers, particularly women, would look for these garages which provided easier parking, or were discouraged from using parking garages altogether.

It seemed that attendant parking was becoming more popular, because it was quicker than self parking. A man could drive up to a garage, leave his car there at ground level to be parked, and immediately walk away ; when he came back for it there was the minimum of delay, because the attendant could produce the car at the exit more quickly than he could.

A point which Mr. Underwood had not mentioned was that, where there was attendant parking, ramp gradients could be steeper, which led to economy. Experienced attendants could drive on inclines of one in five whilst the public at large probably required one in ten.

There was considerable difference in U.S.A. between the older and the more modern types of garage in respect of appearance. The older ones were usually depressing in the extreme. Mostly they were fully clad and, because they were constructed as cheaply as possible, they were poor aesthetically. The more recent ones, which were similar to the example which Mr. Underwood had described, were more successful. Even more attractive, he suggested, was the completely open type with no walls where vehicles were prevented from running over the edge by something almost invisible ; steel cables were very popular for this purpose. This type of garage was light in appearance and colourful because of the cars which were plainly seen—and after all, cars were washed more often than the exteriors of buildings.

In San Francisco, where conditions were very similar to those in our own congested cities, the building of a parking garage usually appreciated the value of property in the neighbourhood and stimulated the owners of adjacent property to redevelop their buildings and bring new commercial life into the area. This was an encouraging aspect of parking garages which was perhaps not well known to developers in this country.

Mr. J. A. Derrington (Associate-Member) referred to Fig. 1 in the paper concerning the efficient use of floor space for parking where the best layout showed an efficiency of just over 60 per cent ; this required 208 square feet of floor per car. Later diagrams showed this layout worked into a structure with a 40 ft. span. If the architect or engineer who had planned that garage had started with the intention of making the structure as cheap as possible and had doubled the number of columns in it, he would have cut the structural cost possibly by 50 per cent. He could have then accepted a less efficient use of floor space and have provided 300 sq. ft. per car at the same cost.

A few years ago a multi-storey garage was erected at Blackpool, accommodating 280 cars. According to his calculations, they used about 71,000 sq. ft. an average of 250 sq. ft. per car, overall. Perhaps the Lancashire drivers were more skilful than those in the south but the column grid in this building was 22 ft. × 24 ft. and no great waste of floor space was made.

Referring to some pictures of precast units which Mr. Underwood had shown in Fig. 3 of the paper, he said one would need to use a five ton crane to pick up some of these units, and that would add up to £20 per car place to the cost of the structure.

Finally he asked that the floor finishes in garage buildings should be given serious consideration. He urged that it was important, when a large area of concrete slab was exposed to weather, to control cracks which would be produced by shrinkage and temperature movement ; and suggested that a rock asphalt would be of value in the solving of this problem.

Mr. Underwood, replying to the question on floor finish, said that throughout the job they were using a high strength concrete. The floor, or deck, was of precast beam units with an infill of the high strength concrete, and the total thickness was 3¾ inch ; 3½ inch was structurally required, and the other ¼ inch was for wearing surface. They had faith in the cross fall to get any water away and he did not think any other precautions would be required on normal decks. In the case of the showroom, however, the greatest precautions had been taken to exclude any moisture from it.

He felt that Mr. Derrington was a little misunderstanding concerning the parts which were to be precast. All the beams and columns were actually cast in situ, and the only parts that were being precast were the floor or deck and the balustrade. The balustrade units were the heaviest, and their weight had been kept down to about 32 cwts., so that they were within the capacity of the normal contractor's tower crane of 80 or 90 ft. radius. The crane was selected at an early stage in the design of the job and all the units were pared down so that they were within the capacity of that crane.

Concerning the percentage efficiencies, it was necessary to fix the width of roadway, and they had arrived at a width of 24 feet. The first diagram in Fig. 1 had a 20 ft. roadway ; it was merely an attempt to show that, under the conditions set down in that diagram, the normal-to-the-kerb type of parking gave the most satisfactory result.

So far as the column spacing was concerned, he felt that everyone would like to see all the columns eliminated. They had put the columns 4 ft. back from the edge of the access road, which gave better manoeuvring into the car bays, and in so doing they had also achieved the most efficient moment position for the beam which extended across the two columns. The 12 ft. cantilevers also gave an opportunity to increase the headroom in the centre.

He had not seen the garage at Blackpool to which Mr. Derrington had referred, but he must make a trip there in order to see whether he could effect further economies at Bristol ; he must do that very soon.

Mr. S. B. Tietz (Associate-Member) asked if the author had carried out or knew of any research which substantiated that a loading less than that suggested by the Code might be reasonable.

Mr. Underwood said that if one spread the loads imposed by cars on the present arrangement one would find that about 40 lb/sq. ft. gave an adequate loading. With the normal-to-the-kerb arrangement there was only half the load of each car on the cantilever ; so that the outer part of the deck could be made very light indeed. The Code loading was fixed, however, not

necessarily just for the overall bending moment, but also for the point loads under the wheels, or jacks. It thus covered all likely conditions of point loading under a " distributed " envelope.

Mr. P. Nuttall (Associate-Member) asked if the Author had comparative figures to show the number of cars taken off the public roads per hour by this type of garage, the ordinary land garage, and the mechanical garage.

Mr. Underwood said he had not comparative figures, but figures for the Corporation open field park in the centre of Bristol, quite close to the site he was concerned with, showed that it could discharge about 400 cars per hour. He thought that the discreet owner-drivers, especially those who had used the Park regularly, would back into position rather than drive nose-in, for that would give them a quick get-away. In the multi-storey park he had described the 24 ft. access road would give space for manoeuvring. One tended to imagine such a park absolutely jammed full, and some driver stalling in the middle of the access road. That did not often happen even on the main roads, and should not therefore be anticipated as a constant hazard in Parks.

With regard to automatic parking, a park described recently had 16 lifts to deal with 500 cars, which worked out at about 30 cars per lift, a very expensive business. If all the owners of parked cars served by one lift turned up within five minutes of each other, some waiting would appear inevitable. The rate of discharge was said to be 12 cars per minute, but presumably this assumed that all the lifts were operating simultaneously.

When using the type of owner-driver park that was described in the paper a driver was his own master. He could operate the passenger lift, proceed directly to the deck on which his car was parked, get into it and drive away, and he would not be half so much irritated as he would be if he had to stand at ground floor and wait for his car to be brought down.

Mr. F. R. Bullen (Vice-President) referred to costs. It seemed to him that it was absolutely essential, if car parks were to be popular with motorists, that the cost of parking should be reduced to a very low minimum. Indeed, he would go so far as to suggest that until we had arrangements whereby the first two hours of parking were free of charge, it would be difficult to persuade motorists to use the car parks. The newspapers on the previous day had mentioned the new car park in the City of London, under the new road, where it was stated that the charge would be 7s. per day. Somebody in the City was complaining because only 40 motorists used it, whereas it was made to accommodate 240 or 280 cars. That situation spoke for itself. One did not blame the motorist ; he could not afford 7s. a day on top of all the other charges.

So that the whole essence of car parking must be economy. There must be a lot of people who would leave their cars for more than two hours, and their payments would repay the operating costs.

One of the most important considerations was to regard the car parks as such ; they were not buildings, and he would have thought that the first thing needed was a new set of by-laws. Car parks did not need windows and walls and painting. What we wanted was an extension of our road system ; that was the ideal and most convenient car park, and it seemed to be the essence of Mr. Underwood's scheme that it was in fact an extension of the road system. It had a great deal to commend it.

Mr. Underwood said Mr. Bullen's remarks reminded him that he had omitted to reply to Mr. Sharp's reference to the fact that in America some multi-storey garages had just the edges of the beams showing from the outside, and that the structures were attractive. Mr. Bullen had given the answer. We had to provide a barrier to prevent cars over-running. The aim was to simplify the structures as completely as possible, we must get down to the bare essentials ; then we could achieve economy. When we had other complications, including fire sprinklers, ventilating systems, and so on, they ran away with the costs.

Lt.-Col. G. W. Kirkland, M.B.E. (Vice-President) said he had not intended to join in the discussion, but after Mr. Bullen's remarks he wanted to say that he was very much opposed to the contention that the motorist had a perfect right to park at the side of the road, where it cost him nothing. Surely that was very selfish.

He joined Mr. Bullen in the view that it was high time the authorities got it into their minds that we were not providing a building to accommodate cars, but a palletised structure.

Mr. W. A. Nicholas (Associate-Member) said the building described in the paper was in reinforced concrete. Nowadays pre-fabrication was becoming more and more popular, particularly on the Continent ; and the contractor preferred to work with precast concrete. For this particular job he thought prefabrication could be used to advantage ; he suggested that some sort of device could be adapted to precast beams or some sort of portals, prestressing where necessary and erecting them so that the shuttering could be fixed to the beams or portals.

Next he made the point that multi-storey car parks had to be located in populous areas, so that the aesthetic aspects must be carefully considered.

The balustrade for the structure at Bristol looked rather complicated. He would have thought that to provide pockets on the main beams and to fix the balustrade to them would have been more economical and perhaps easier.

Mr. Underwood replied that the materials with which the car park was constructed, of course, were very much controlled by the Regulations. It could easily have been of steel framing which, when erected, would then have needed to be clad with concrete, but it would have involved greater expenditure than would the method that was actually used. If the Regulations ever premitted the use of naked steel in such a structure he thought there would be a very strong case indeed for steel framing with concrete decking etc.

Concerning the reference to precasting the units, he said the contractor had been working with his firm as a team since very early on in the design stage, and they had discussed the matter thoroughly. After considering the weight of the portals or frames they decided not to precast, but to have form-work which could be made in a very permanent sort of way.

If they had precast the beams they would have had comparatively few heavy lifts for a crane, and it was extremely costly to have heavy cranes on a site which were to be used most of the time for light work ; it was like employing a man to do a boy's job. So that he thought the choice made, to carry out all the main framing in situ, was the right one, using the lighter type of crane with a fairly big radius, about 90 feet, for hoisting the materials, pre-cast or otherwise to any part of the job.

He would very much like to have a steel frame building when the regulations permitted its more economic use.

Mr. D. N. Mitchell (Member) agreed with the comment that multi-storey car parks should be considered by the designer not as buildings but as parking spaces.

However, the proposal to use uncased structural steelwork did not appeal to him from the point of view of fire risk. These structures should be designed as economically as possible, but we must not forget that people have to go into and come out of them with safety.

Mr. UNDERWOOD agreed completely. The situation with regard to the protection of steel was under review and there were many devices being considered which would remove a good deal of the criticism of the use of steel in buildings without the very heavy load of the normal concrete encasement. He thought that when that time came a composite construction would be used, using the normal reinforced concrete columns, with steelwork, properly treated, for the spans. That would mean quite an advance in economy.

Mr. PETER BOSTON said he had read an article in October in "The Financial Times," written by a director of Lex Garages, which referred to a structure almost identical with that described in the paper. Figures given in this article indicated that the cost per car considerably exceeded that applicable to an automatic stacking garage in spite of the fact that the writer seemed to have been able to park twice as many cars in a given space as Mr. Boston had been able to do working on space allotments similar to those used in the Bristol car park. Mr. Boston asked if the actual costs per car, including all the ancillaries, could be given for the Bristol car park.

Recalling a reference by a speaker to a ramp, he said the aesthetics were perfectly satisfactory; classical paintings of the tower of Babel, showing a higher version of the same thing, were acclaimed all over the world.

Mr. UNDERWOOD said the answer with regard to costs was that the particular article referred to gave a hypothetical example, based on a projected site in the City of London; the cost of the site was assessed at £200,000, which was very high in comparison with the cost of the structure.

In his own figures for the Bristol car park he had included for the showrooms, the central filling station, and so on. It had been possible to let the building to the tenant at a price per car bay which was very low indeed; it was based on the fact that the showroom accommodation and the petrol filling stations justified higher rents than the car park structure only.

He thanked Mr. Boston for his remarks on the aesthetics.

Mr. B. L. CLARK (Member) said that he also had studied car parking and it was a most interesting subject.

Following the earlier remarks about using the streets as car parks, he considered that the attitude towards car parks was quite wrong. We might as well build railways without engine sheds as roads without facilities for parking cars. When one bought a car, one spent a lot of money on purchase tax, and so on, and there was also the tax on petrol. There was a vast industry manufacturing motor cars, one of the biggest manufacturing industries in this country, employing very many thousands of people. Were we to kill the use of motor cars in our cities by making the situation so difficult? We must have a more broad-minded attitude towards car parking, and, as Mr. Bullen had said, we should consider car parks as an extension of our road system, and try to get the Government to accept them in that way. We wanted a new angle of view on the matter; let us make the car parks look like roads and try to get the motorist to pay for them.

Mr. UNDERWOOD said the first approach made with this idea of the multi-storey car park was to the Ministry, when all the points were stressed and it was pointed out that it could form part of a road system. But the Ministry's declared policy, of course, was that it was a local problem and must be solved by the local Authorities.

THE PRESIDENT, closing the meeting, thanked Mr. Underwood for the excellent way in which he had answered the questions raised.

Book Reviews

Frames and Arches. Condensed Solutions for Structural Analysis by Valerian Leontovich. (New York and London: McGraw-Hill, 1959). 9 in. × 6 in., 472 + ix pp. £7 15s.

Valerian Leontovich has presented this volume of condensed solutions for the structural analysis of twenty types of frames and arches in the characteristic style so successfully employed by Prof. A. Kleinlogel. The work covers symmetrical pinned and fixed base portal frames of rectangular, trapezoidal, pitched and parabolic profile of both constant and variable moment of inertia, and provides a series of simple equations leading to the solution of such frames when subject to a variety of external loading conditions.

Where the volume deals with frames of constant cross section, it differs very little in the operations for solution from Kleinlogel although the latter covers a rather greater number of frames. It does, however, offer formulae for the solution of frames and arches of variable moment of inertia and it will be with this type of frame that the engineer will find it to provide a useful tool for rapid solution, particularly for those called upon to solve such frames regularly. For the more casual user, the need might be felt to seek rather regular guidance from the worked examples given, and column analogy might well provide as rapid an answer for pinned ended frames of straight members of variable cross section, as reference to the charts and tables in the book and substitution of values in the fifteen or so simple expressions required to evaluate all the bending moments and forces. However, the more involved parabolic arch frames, and frames with fixed end conditions having variable cross section, would not very readily be solved by methods other than the condensed solutions given, and the volume may commend itself to engineers on this account.

The book also provides, in tabular form, dimensional data of parabolic tapered arches and formulae for the calculation of the effects of axial deformation for use when the rise to span ratio exceeds 0·2.

It might have been advantageous for a loose leaf form of the explanation of symbols and the common general frame constants to be included to facilitate cross reference. R.P.H.

The Midland Soil Mechanics and Foundation Engineering Society Proceedings, Vol. I, 1957. (University of Birmingham, Dept. of Civil Engineering, 1959). $10\frac{1}{2}$ in. $\times$ $8\frac{1}{2}$ in., 139 pp.

This volume is made up of the following papers: "Practical Examples of Bearing Capacity Calculation," by T. K. Chaplin "Limit Design of Flexible Walls," by P. W. Rowe, "The Influence of Site Investigation on Construction Methods," by A. Little, "Stability of Graded Filters," by J. Kolbuszewski, "Shear Strength and Pore Pressure," by A. D. M. Penman, "The Allowable Settlements of Buildings," by A. W. Skempton and "Aspects of Softening in Clay," by T. K. Chaplin.

Proceedings of the Second Japan Congress on Testing Materials. Compiled by the Editorial Committee of Japan Congress on Testing Materials with co-operation of the Science Council of Japan. (Kyoto, Japan, 1959). 11 in. $\times$ $8\frac{1}{2}$ in., 244 + ix pp.

Several hundred specialists took part in this Congress held in Kyoto in October, 1958, at which 119 papers were presented on a wide variety of subjects. 67 of these papers are printed in this volume including the following: "On the Brittle Fracture of a Mild Steel under repeated impacts," "On the methods of making concrete test cylinders," "An analytical testing method for proportion of hardened concrete," "The effect of mix proportions on the frost resistance of concrete," "Static and dynamic moduli of elasticity of concrete," "On the measurement of loss in ultrasonic pulse in concrete," "Studies in permeability of mass concrete," "On the secondary consolidation of clay" and "A study on strength characteristics of over-consolidated clay by means of triaxial tests."

Petroleum Refinery Manual, by H. M. Noel. (New York: Reinhold Publishing Corporation; London: Chapman & Hall, 1959). 10 in. $\times$ 7 in., 182 + xi pp., 64s.

The object of this book is to give an integrated picture of the whole process of petroleum refinery design and construction. It includes descriptions of all the standard techniques of petroleum processing and gives information on planning and constructing new refineries, including estimating, scheduling, shop fabrication and field construction. Examples of finished work are described, and figures covering capital investment, operating manpower and operating costs accompany the descriptions and diagrams of modern refining processes.

Proceedings of a Symposium on the Strength of Concrete Structures, London, 1956. (London: Cement and Concrete Association, 1959). $9\frac{1}{2}$ in. $\times$ 6 in., 697 + vii pp., £5.

The papers given at a Symposium organized by the Cement and Concrete Association in conjunction with the Joint Committee on Structural Concrete are published in this volume, together with the general reports and discussions. The main object of the Symposium was to provide an opportunity for those interested in the design of reinforced and prestressed concrete structures to exchange information and opinions regarding methods of design, with particular reference to the ultimate load theory, and there were contributions from some overseas specialists as well as British papers.

The seventeen papers delivered at the Symposium were divided into five sessions, of which the first, under the Chairmanship of Dr. W. K. Wallace, was devoted to a consideration of factors of safety. Dr. M. R. Horne in "Some results of the theory of probability in the estimation of design loads," dealt primarily with the treatment of loading intensities, investigating the probability approach to a number of loading questions. The second paper, "The determination of the design factor for reinforced concrete structures," by Dr. Arne I. Johnson, considered economic factors as well as the strength of materials and the magnitude of loads in the method of design, and Professor Sir Alfred G. Pugsley in his paper, "Current trends in the specification of structural safety" dealt with the all-over problem of structural safety.

Professor R. H. Evans was the Chairman for Session B and Professor A. D. Ross gave the general report. The five papers delivered were: "The strength of singly reinforced beams in bending," by Dr. A. H. Mattock, "The strength of concrete members in combined bending and torsion," by Mr. S. Armstrong, "The strength of prestressed concrete members," by Professor C. P. Siess, "Moment redistribution in continuous beams reinforced with plain and deformed bars," by Dr. H. Kajnal-Kónyi and Mr. H. E. Lewis and "The failure of concrete under compound stress," by Mr. A. J. Harris. Three papers dealing with statically indeterminate structures made up Session C. These were: "Ultimate load design of reinforced and prestressed concrete frames," by Professor A. L. L. Baker, "The strength of statically indeterminate prestressed concrete structures," by M. Y. Guyon and "The strength of prestressed concrete continuous beams and simple plane frames," by Professor P. B. Morice and Mr. H. E. Lewis. The general report was given by Mr. A. Goldstein, the Chairman for the session being Professor A. J. S. Pippard.

Session D, under the Chairmanship of Mr. S. Vaughan, contained the following three papers: "The strength of concrete walls under axial and eccentric loads," by Mr. A. E. Seddon, "The strength of concrete members under dynamic loading," by Dr. S. C. C. Bate and "The strength of simply supported slab bridges subjected to concentrated loads," by Professor P. B. Morice and Mr. G. C. Reynolds. The last Session, for which the Chairman was Dr. A. R. Collins and the general reporter Dr. F. G. Thomas, was devoted to the application of the researches made and the views expressed to methods of design in codes of practice, etc. The papers included were: "Ultimate strength of reinforced concrete in American design practice," by Dr. E. Hognestad, "The design of reinforced concrete members," by Dr. A. Aas-Jakobsen, which described a method of design, which, in part, formed the justification for the relevant clauses in some European codes and the A.C.I. code for 1951, and included recommendations not yet published in a code, and "Load factor design in building regulations: future British practice," by Dr. D. D. Matthews.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 24th March, 1960, at 5.55 p.m. Mr. Lewis E. Kent, B.Sc., M.I.Struct.E., M.I.C.E., (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

STUDENTS

BANERJI, Narayan Chandra, of London.
BRAND, Kenneth Frederick, of Hutton, Essex.
CHAN SIEW FONG, of Singapore.
FITZPATRICK, John Anthony, of London.
GALLANNAUGH, Roger, of Burnham-on-Sea, Somerset.
HENDERSON, James, of Sherborne, Dorset.
HORTON, Brian, of Surbiton, Surrey.
JACKSON, Edward John, of Halstead, Kent.
JUST, David James, of Birmingham.
LEE CHEOK HOE, of Singapore.
MARRIOTT, Kenneth, of Alfreton, Derbyshire.
MORLEY, Peter Stuart, of Allestree, Nr. Derby.
NYE, Colin, of Pontefract, Yorkshire.
PAO WEI KANG, of Hong Kong.
PORTER, Colin Allan, of New Zealand.
SLAUGHTER, David Anthony, of Birmingham.
STEPHEN, Graham Meredith, of Rogerstone, Nr. Newport, Mon.
TANG NGI WAH, of Singapore.
TAYLOR, Howard Peter John, of Iver Heath, Bucks.
TOWLER, Malcolm Philipson, of Liverpool

GRADUATES

APPLETON, Reginald Howard, of Chelmsford, Essex.
ATKINSON, Edward Peter, B.Sc., of Sheffield.
BASU, Tapan Kumar, B.Eng., of London.
BIYIKOGLU, Mehmet Ali, of London.
BUCH, Jitendra Dolarrai, B.E., of Bombay, India.
CLAPHAM, Geoffrey Smith, A.M.I.Mun.E., of Bradford.
CLUDERAY, Colin, of Leeds.
DAY, Thomas Michael, of Norwich, Norfolk.
EATON, Malcolm, of London.
EMERSON, Geoffrey Joseph, B.E., of Galway, Ireland.
GABRIEL, Rodney Frank, of Feltham, Middlesex.
GODFREY, Anthony, of Swanley, Kent.
GRIMSTON, Reginald, of London.
KING, John Kenneth, of Brighton, Sussex.
LEADBETTER, Kenneth Edward, B.Sc., of Stockport, Cheshire.
LIM CHEE MENG, of Kuala Lumpur, Malaya.
MAKEPEACE, Brian Humphrey Miles, of London.
MATTHEWS, Douglas Thomas, of Bebington, Cheshire.
PAL, Sudhindra Nath, B.Eng., of Calcutta, India.
PARASHARAMI, Ashok Gopal, B.Eng., of Poona, India.
PEARCE, Donald David, of Widnes, Lancs.
POAD, John Gordon, of Stockton-on-Tees, Co. Durham.
RAY, Syamal Kumar, B.Sc., of Watford, Herts.
REYNOLDS, John Edward, of London.
SHERIFF, Donald, of Sheffield.
SIDDIQUI, Abdul Ali, B.Eng., of Karachi, Pakistan.
STOCKWELL, Malcolm Lee, of Ormesby, Middlesbrough.
STUDER, Cecil Robert, of Farnham, Surrey.
THOMPSON, Victor George, of Welling, Kent.
WHITWORTH, Kenneth Malcolm, B.Sc.,Tech. of Rochdale, Yorks.
WILLCOCKS, James Peter, B.Sc., of London.

MEMBERS

CROMBY, John, of Port Swettenham, Malaya.
LE SEELLEUR, Col. Alfred John, O.B.E., of Purley, Surrey.
SMITH, Hubert John Evans, B.A., A.M.I.C.E., of Burgess Hill, Sussex.

TRANSFERS

Students to Graduates

BEALE, Francis Curteis, of Dorking, Surrey.
GOSLING, Ernest Harvey, of Toronto, Ontario, Canada.
HAIGH, Melvin, of Wakefield, Yorkshire.
HAYES, Michael Charles, of Croydon, Surrey.
MACKENZIE, Peter John, of Bath, Somerset.
WAINWRIGHT, Malcolm Stanley, of Ellesmere Port, Cheshire.
WARDEN, John Charles, of Hornchurch, Essex.

Graduates to Associate-Members

CHARMAN, William Royston, of Hildenborough, Kent.
DEAN, Charles Michael, of Shepperton, Middlesex.
HUNT, Dennis William, of London.
WELLINGS, Paul Eustace, of Bishop's Stortford, Herts.

Associate-Members to Members

CLARK, James Alexander, A.M.I.C.E., of Glasgow, Scotland.
FIRMINGER, Leslie Alfred, of Sutton Coldfield, Warwickshire.
HORTON, John Pearson Guy, of Streetly, Staffs.
PIKE, Cecil Wilfrid, A.M.I.C.E., of Woodmansterne, Nr. Banstead, Surrey.
SLATER, Alfred, of Auckland, New Zealand.
SMULLEN, Herman, of London.
SPIRA, Ephraim, of Addis Ababa, Ethiopia.
STORY, John Bower, of Warrington, Lancs.

Members to Retired Members

PAYNE, Paul Graham, of Shanklin, Isle-of-Wight.
WARNOCK, John, M.C., M.I.C.E., of Delanwy, North Wales.

Associate-Member to Retired Associate-Member

RAMAPPA, Nivarti, of Anantapur Andhra Pradesh, India.

OBITUARY

The Council regret to announce the deaths of Arthur MONK (Member), Arthur ASHTON, John Goodrich KAY (Retired Members), Jose Antonio DENIZ, Frederick HADFIELD (Associate-Members), Richard Owen RADFORD (Graduate).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of Herbert Lawrence BRIGGS, Kenneth Austin PHILLIPS (Graduates), Michael Ross EADIE (Student).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, 26th May, 1960.

Ordinary General Meeting for the election of members at 5.55 p.m., followed by the Annual General Meeting at 6 p.m.

Thursday, 23rd June, 1960.

Ordinary General Meeting for the election of members at 5 p.m.

INSTITUTION OF STRUCTURAL ENGINEERS BENEVOLENT FUND.

The Annual General Meeting of the Voting Contributors to the Institution of Structural Engineers Benevolent Fund will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 26th May, 1960, at 6.20 p.m.

EXAMINATIONS—JANUARY, 1960.

HOME AND OVERSEAS CENTRES

The Examinations of the Institution were held in January, 1960, at the usual centres in the United Kingdom and Overseas at the following centres :—

Accra, Aligarh, Auckland, Barbados, Beirut, Bethlehem (South Africa), Bombay, Bulawayo, Cairo, Calcutta, Cape Town, Colombo, Dacca, Dar-es-Salaam, Durban, East London (South Africa), Hong Kong, Jesselton, Johannesburg, Khartoum, Kingston (Jamaica), Kuala Lumpur, Lagos, Lahore, Madang, Madras, Mauritius, Montreal, Nairobi, Ndola, Rangoon, Salisbury, (Southern Rhodesia,) Singapore, Sydney, Toronto, Trinidad, Wellington, Windhoek and Winnipeg.

One hundred and eighteen candidates took the Graduateship Examination (74 at home centres and 44 overseas) and 600 took the Associate-Membership Examination (474 at home centres and 126 overseas). Of these, 53 passed the Graduateship Examination (27 at home centres and 26 overseas), and 111 passed the Associate-Membership Examination (90 at home centres and 21 overseas). The names of the successful candidates are :—

Graduateship Examination

Atkins, Walter Arthur
Bagchi, Rabindranath
Basu, Mrinal Kanti
Ben Souda, Ahmed Ben Dris
Bharadwaj, Joginder Paul
Bobisch, Gerhard
Buder, Domenico
Carrick, John Leslie
Carrington, Vernon
Carson, Samuel William
Creet, Armen John
Davis, Victor Clayton Aliston
Duffy, Patrick
Ecob, John Russell
Fatehi, Abdulkadir Abdulhasan
Finch, Victor Leopold
Gerrard, John Roy
Gifford, Robert Charles
Gregoriades, Costas Gregory
Griffiths, Brian John
Gyani, Kishor Ranchhodlal
Harel, Georges Alfred Pierre
Holman, Charles Edward
Kirkland, George John
Korevaar, Willem Huibertus
Leiman, Bernard
Leithead, Gordon McLean
Levy, John
Lim Chwee Bok
McCarthy, Patrick Dennis
Makin, William Alan
Marghetis, Christos Constantin
Maujean, Joseph Lucien
Ososami, Israel Adebayo
Parikh, Punamchand Shankerlal
Parker, Joseph Martin
Pitchers, John
Purkayastha, Ramapada Kar
Rane, Abaji Sadeshiv
Reed, John Barton
Richter, Colin Philip
Rowe, Walter Ernest
Schroeder, Bryan Payne
Shears, Peter John Randal
Shilling, Michael Frank
Smith, Anthony Lloyd
Tatu, Shreekant Gajanan
Taylor, John
Tilbury, John Alan
Unger, Ralph Richard
Webb, Graham Charles
White, John Ronald
Whitmore, Richard Graeme

Associate-Membership Examination

Acton, Ian Murray
Alderson, John Anthony
Allott, John Thomas
Amodia, Rajnikant Mulchand
Annells, Michael
Atkar, Dilbag Singh
Baynham, Wilfrid
Bogle, John Brian
Botterill, John Alfred
Brown, David Robert Matthew
Brown, Peter Allan
Burchess, David Sydney
Burgess, Kenneth Arthur
Butler, Alan
Butler, Robert
Cannell, Gordon John
Chan Kai-Ming
Chanda, Sankar Lal
Chorley, Victor Wilfred
Clayden, Kenneth John Charles
Daugherty, Frank
Dhar, Paramesh Ranjan
Divan, Malhar Shivaram
Ducker, James
Fox, Dennis
Fozzard, John
Gabay, Leslie Oliver St. Clear
Garrad, Reginald John
Ghosh, Utpal Krishna
Gill, Michael Brendan Mary
Girardier, Edward Victor
Gizejewski, Krystian
Gray, Walter Smith
Griffiths, Edward John
Harkin, Michael Edward
Harris, Edward George
Harris, George Graham
Heard, Michael John
Hewitt, Lawrence John

HEWSON, Kenneth Norman
HOLLINGTON, Michael Robert
HOMERSHAM, Peter Gerald
HUGALL, John Eric
HURDEN, Michael Eric
JAHINA, Kaio
JAMES, Arthur Neville
JONES, Chandler
JONES, Michael Sidney
KEYS, Gerald
KHADILKAR, Bhaskar Shrikrishna
LAMBERT, Frederick Walter
LEE, John Kenneth
LEE, Terence Godfrey
LIMBADA, Ismael Ahmad
LOAT, Reginald Dennis
LONDHE, Vishwas Dattatraya
LOWE, John
LYTH, Philip Bernard
MCARDLE, Brian
MCFARLANE, John
MACLACHLAN, Ian Hamilton
MALONE, Patrick Oswald
MAYCROFT, Douglas Harvey
NEWSOME, Brian
O'BRIEN, Leo
O'BRIEN, William Rex
ORAON, Kartik
OSBORNE, Ivan William
PATEL, Janakray Rambhai
PESKETT, George John
PHILLIPS, William
PYE, Ralph
RIDLER, John Walter Arthur
ROBSON, Keith
ROBSON, Ronald John
ROSSINGTON, Antony Bevan
ROWBOTHAM, Noel Gervais
RYMILL, Frederick Michael
SCHEUER, Marius Cornelis
SEARLE, Michael George
SESHAN, Tattamangalam G. I.
SHAH, Chandrakant Bhailal
SHARLAND, Brian Walter
SINGH, S. Balbir Bhambroy
SINHA, Thakur M. N.
SLACK, George Edwin
SLANEY, John Richard
SMITH, David Harry
SNELL, John Alfred
SNELL, Martin de Putron
SPACEY, Geoffrey
STAPLES, Graham George
STAPLETON, Roger Louis
STUART, Allan
SULLEY, William David
TAWDE, Ashok Nanasahib
TAYLOR, Wilfred Samuel
THORPE, Edwin Holt
TILBURY, Frank Charles
TONGE, Derek George
TOPPING, Jack
TOWSON, Valentine Geoffrey
TRUSSLER, Stanley
TURKINGTON, William Kenneth Somme
TWIGGER, Michael Arnold
VITHAL, Nori Panduranga
WAITE, Dennis
WARD, Albert Thomas James
WHITTAKER, James Roy
WONG, Kai Cheong Franklin
WYATT, Albert Frank

EXAMINATIONS, JULY, 1960

The Examinations of the Institution will next be held in the United Kingdom and overseas on Tuesday and Wednesday, July 12th and 13th, 1960 (Graduateship) and Thursday and Friday, July 14th and 15th 1960 (Associate-Membership).

MECHANICAL HANDLING EXHIBITION

The Organisers of the Mechanical Handling Exhibition and Materials Handling Convention have extended an invitation to members of the Institution to visit the Exhibition which is to be held at Earls Court, London, from the 3rd to 13th May, 1960. Tickets of admission may be obtained upon application to the Secretary of the Institution.

HONOURS AWARD

In offering their sincere congratulations to the following member on the distinction conferred upon him, the Council feel that they are also expressing the good wishes of the Institution.

Order of the British Empire—O.B.E.
Mr. J. Owen Parry (Member).

INSTITUTION AWARDS

The Council have awarded the Institution Bronze Medal for the Session 1958-59 to Mr. Derek Bond for a paper, "Design, Model Analysis and Testing of an 83 ft. span Interconnected Portal Grillage."

The Institution Branch Award (for the best paper from amongst those for which Branch Prizes have been awarded) has also been awarded to Mr. Bond for this paper.

Mr. W. S. Watts has received the Council's commendation for his paper, "Some Points of Structural Interest at Calder Hall 'A' Nuclear Power Station."

REPRESENTATION

The Council have appointed the following Institution representatives :—

Manchester and District Advisory Council for Further Education—Post Advanced Building and Civil Engineering Advisory Council
Mr. J. E. Guest (Member).

North Western Regional Advisory Council for Further Education—Joint Advisory Committee for Education in the Building Industry
Mr. W. Fitton (Member of Council). (re-appointment),

Joint British Committee for Stress Analysis
Professor P. B. Morice (Associate-Member of Council).
Dr. E. Lightfoot (Delegate Member of Council).

OVERSEAS REPRESENTATIVE

The Council have appointed Professor J. W. Roderick (Member), Institution Representative in Victoria and New South Wales, Australia, for a period of five years in place of Professor A. J. Francis, who has relinquished the appointment.

YEAR BOOK AND LIST OF MEMBERS

The Year Book and List of Members for 1960 will go to press in July, for publication in October, when a copy will be sent to all members.

Members are requested to inform the Secretary of any alterations in titles, degrees or addresses, which have not already been notified, by June 27th, in order that such amendments may be included in the new edition.

ADDITIONS TO THE LIBRARY

Bannister, A. and Raymond, S. *Surveying*. London, 1959. Presented by Mr. H. H. B. Stewart.

Benson, C. S. *Advanced Structural Design*. London, 1959. Presented by the Author.

Clendinning, J. *Principles of Surveying*, 2nd Edition. London, 1960.

D.S.I.R. *Principles of Modern Building*, Vol. 1, 3rd Edition. London, 1959.

L'Hermite, R. *Méthodes Générales d'Essais et de Controle en Laboratoire*. Livre I—*Mesures géométriques et mécaniques*, Paris, 1959. Presented by Mr. P. J. Gerard.

Japan Congress on Testing Materials. *Proceedings of Second Congress on Testing Materials*. Kyoto, 1959.

Japan Society of Civil Engineers and Architectural Institute of Japan. *Proceedings of the Symposium on the Failure and Defects of Bridges and Structures*, Tokyo, 1958.

The Midland Soil Mechanics and Foundation Engineering Society Proceedings, Vol. I, 1957. University of Birmingham, 1959.

Noel, H. M. *Petroleum Refinery Manual*. New York and London, 1959.

Stubbs, F. W. (Editor) *Handbook of Heavy Construction*. New York and London, 1959.

Symposium on Strength of Concrete Structures, 1956, *Proceedings*. London, 1959.

Tuma, J. J., French, S.E. and Lassley, T. I. *Analysis of Continuous Beam Bridges*, Vol. 1. *Carry-Over Procedure*. Oklahoma State University, 1959.

Branch Notices

LANCASHIRE AND CHESHIRE BRANCH

The Annual Dinner Dance will be held on Friday, 6th May, 1960, at the Carlton Club, Eberle Street, Liverpool.

Hon. Secretary: W. S. Watts, A.M.I.Struct.E., A.M.I.C.E., 11, Newchurch Lane, Culcheth, Nr. Warrington, Lancs.

MIDLAND COUNTIES BRANCH

Hon. Secretary: S. M. Cooper, A.M.I.Struct.E., "Applegarth," Hyperion Road, Stourton, Nr. Stourbridge, Worcestershire.

GRADUATES' AND STUDENTS' SECTION

Third Annual Buffet Dance will be held on Friday, 6th May, 1960, at the Station Hotel, Dudley.

Hon. Secretary: H. T. Dodd, Shepherd's Cottage, Grove Lane, Wishaw, Sutton Coldfield, Warwickshire.

NORTHERN COUNTIES BRANCH

Hon. Secretary: P. D. Newton, B.Sc., A.M.I.Struct.E., A.M.I.C.E., c/o Richard Hill Ltd., P.O. Box 29, Middlesbrough, Yorkshire.

NORTHERN IRELAND BRANCH

Hon. Secretary: L. Clements, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., 3, Kingswood Park, Cherryvalley, Belfast.

SCOTTISH BRANCH

Hon. Secretary: W. Shearer Smith, M.I.Struct.E., A.M.I.C.E., c/o The Royal College of Science and Technology, George Street, Glasgow, C.1.

SOUTH WESTERN COUNTIES SECTION

Secretary: C. J. Woodrow, J.P., "Elstow," Hartley Park Villas, Mannamead, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

Annual General Meeting will be held on Wednesday, 4th May, 1960, at the Mackworth Hotel, High Street, Swansea.

Hon. Secretary: K. J. Stewart, M.I.Struct.E., A.M.I.C.E., 15, Glanmor Road, Swansea.

WESTERN COUNTIES BRANCH

Hon. Secretary: A. C. Hughes, M.Eng., A.M.I.Struct.E., A.M.I.C.E., 21, Great Brockeridge, Bristol, 9.

YORKSHIRE BRANCH

Hon. Secretary: W. B. Stock, A.M.I.Struct.E., 34, Hobart Road, Dewsbury, Yorks.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary: A. E. Tait, B.Sc., A.M.I.Struct.E., A.M.I.C.E., P.O. Box 3306, Johannesburg, South Africa.

During weekdays, Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary: J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P.O. Box 932, Durban.

Cape Section Hon. Secretary: R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

EAST AFRICAN SECTION

Chairman: R. A. Sutcliffe, M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

Hon. Secretary: K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

NIGERIAN SECTION

Chairman: J. W. Henderson, E.R.D., B.Sc., M.I.Struct.E., M.I.C.E.

Hon. Secretary: A. Brimer, A.M.I.Struct.E., Brimer, Andrews and Nachshen, Private Bag Mail 2295, Lagos, Nigeria.

SINGAPORE AND FEDERATION OF MALAYA SECTION

Chairman: T. F. Lee, B.Sc.

Hon. Secretary: W. N. Cursiter, B.Sc., A.M.I.Struct.E., A.M.I.C.E., c/o Redpath Brown & Co. Ltd., P.O. Box 648, Singapore.

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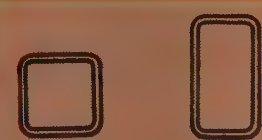

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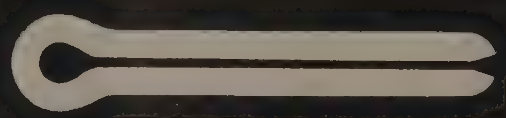

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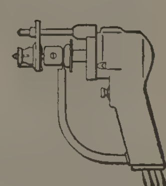

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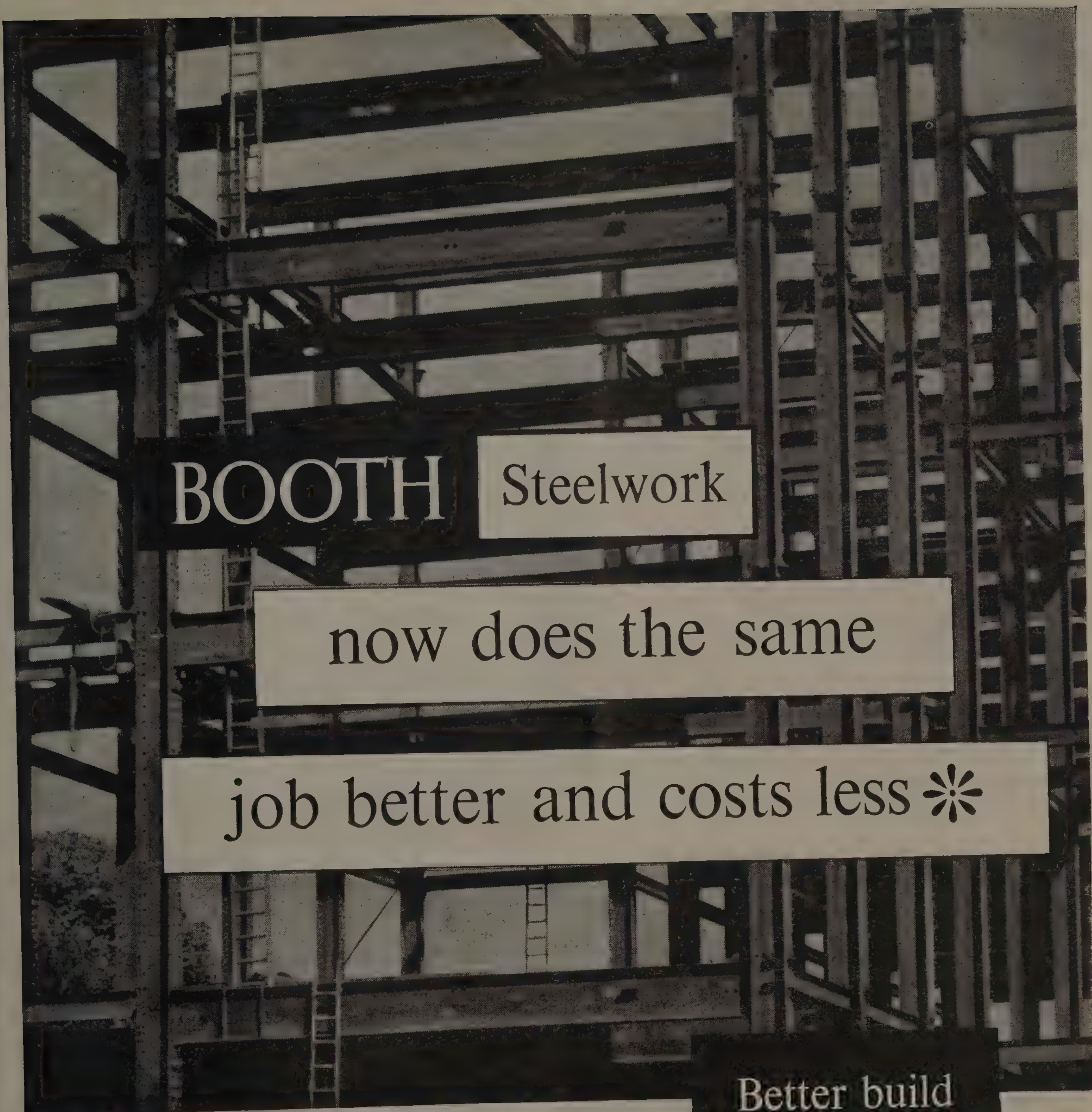
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OFFICIAL APPOINTMENTS

BURSARIES IN STRUCTURAL STEELWORK

Bursaries provided by the British Steel Producers Conference and The British Constructional Steelwork Association are available for the Postgraduate Course in Structural Steelwork dealing with the Application of Modern Analytical and Design Methods to practical design problems.

Students will take appropriate parts of the postgraduate course in engineering structures which will include : analytical techniques for highly indeterminate systems ; theory of elasticity and its application to stress distribution problems, bending and torsion of beams, bending of plates ; buckling of columns, beams, plates and plate stiffener combinations, basic principles of design ; plasticity including limit analysis theorems, collapse load calculations, displacement calculations, effects of shear and axial loads, plastic stability of struts and frames, shakedown analysis, minimum weight design ; photo-elastic and other laboratory techniques ; laboratory experiments to illustrate the above topics.

The design side of the course will include design of connections (riveted, welded, bolted), components (struts, ties, rolled beams, light gauge components, plate girders, vierendeel girders, castella beams), ultimate strength, fatigue behaviour, load factors, safety concepts ; elastic simple semi-rigid, rigid, composite and plastic design of single and multi-storey frames ; design of bridges, towers, gantries, bins and tank structures. The course will deal with the administration of steelwork design, economics, contract documents, relation between client, designer and contractor, organisation of work, responsibilities and liabilities. Drawing office work will include design projects and reports. In addition to laboratory work, opportunities exist for observing the actual behaviour of building structures under construction at the College.

The value of the Bursaries is from £475 to £775 from which fees (£64) will be payable, for the period from October, 1960 to July, 1961. Applications must have a good first degree in Civil Engineering or equivalent qualifaction and preferably some practical experience in structural steelwork design. Applications including the names of two referees should be made to the Registrar, Imperial College, London, S.W.7, by June 1st, 1960.

LEADING DRAUGHTSMEN

required in the Civil and Structural Engineering Branch in the WAR DEPARTMENT Directorate of Works at Chessington. Must be British of British Parentage and have the O.N.C. or equivalent and at least four years practical and drawing office experience in two or more of the following subjects in any section ; a knowledge of surveying and levelling is desirable in all sections.

(a) *Structural Section.*—Reinforced concrete and/or Structural Steelwork design and general building construction-relevant Codes of Practice and British Standards. Design in reinforced concrete includes simply supported and continuous slabs and beams including shear and compression reinforcement, columns, simple frames, foundations retaining walls and load bearing walls. Design in structural steelwork includes bolted, riveted and welded beams, stanchions, frames, trusses, etc., in hot-rolled and tubular steel ; foundations and load bearing walls.

(b) *Civil Engineering Section.*—Building and Civil Engineering construction and design layout and calculation of external services for Housing Estates, Barracks and Industrial Areas. Relevant British Standards and Codes of Practice. Design includes water supply, sewerage, roads, sports grounds, simple reinforced concrete and steel structures, pumping stations, soil mechanics and concrete technology.

(c) *Defence Section.*—Internal structural specialised details within Marston and other standard forms of shedding—reinforced concrete detailing and bending schedules—brick and concrete buildings external services and excavations.

Starting pay in accordance with age, qualifications and experience. Prospects of establishment and promotion. Five day week of 42 hours including meal intervals ; canteen facilities. Salary : Leading draughtsman £895 to £1,055. State age, full details of training and experience and which section applied for to W.D.A.2(c) Directorate of Works, Chessington, Surrey, or any Employment Exchange, quoting Westminster 1064.

OFFICIAL APPOINTMENTS—continued

BRADFORD INSTITUTE OF TECHNOLOGY

Principal: E. G. Edwards, Ph.D., B.Sc., F.R.I.C.

Applications are invited for the post of :—

Principal Lecturer in Civil Engineering

Candidates should possess the qualifications and experience to direct research and post graduate work and to take a leading part in advanced teaching. Corporate membership of at least one of the professional Institutions associated with Civil Engineering and extensive professional experience are essential. The successful candidate will be expected to assist the Head of Department with the administrative work of the Department.

Salary Scale, £1,750 to £1,900 per annum.

Previous industrial and research experience at a suitable level will be taken into account in fixing the commencing salary.

Further particulars and forms of application may be obtained from the Registrar, Bradford Institute of Technology, Bradford, 7.

SUNDERLAND EDUCATION AUTHORITY

The Technical College

DIPLOMA IN TECHNOLOGY SANDWICH COURSE IN CIVIL ENGINEERING

Applications are invited from prospective students for the 1960-61 Session. The course is of Honours Degree Standard, carries the full recognition for the Institution of Civil Engineers, and involves attendance at the College for six months in each of four years. Intervening periods are spent in Industry, many firms having agreed to co-operate in the provision of practical training. Most local Authorities will award grants on the scale used for undergraduates.

Further particulars may be obtained from the Registrar, The Technical College, Sunderland, Co. Durham.

SURREY COUNTY COUNCIL

Applications are invited for the following appointments :—Structural Engineer, Grade IV (£1,065 to £1,220 p.a. plus £30 p.a. London Allowance). Must be qualified Structural or Civil Engineer and have had experience in design and detailing of steelwork and/or reinforced concrete for medium to large scale contracts.

Structural Engineering Assistant, Grade II (£765 to £880 p.a. plus up to £30 p.a. London Allowance according to age). Must have knowledge of detail and some design in steel or concrete.

Candidates will be appointed at the appropriate point within the scale according to age and ability.

Full details, present salary and three copy testimonials to County Architect, County Hall, Kingston, as soon as possible.

ENGINEERS

required in Architect's Department, L.C.C., up to £1,135 (under Review).

(a) *Structural Engineering Division.*—Extensive programme includes multi-storey flats, schools, offices, warehouses and other buildings.

(b) *Structural Engineering Division.*—To deal with applications in respect of special structures under Part IV of London Building Act (A) Act, 1939, and constructional by-laws. Interesting work in involving consideration of unconventional forms of construction.

(c) *District Surveyor's Service.*—Work mainly outside involving negotiations with Architects, engineers and Surveyors and supervision of works in progress.

Good prospects in all three aspects of the Council's work.

Application form and particulars from Hubert Bennett, F.R.I.B.A., Architect to Council (EK/SE/495/5), County Hall, S.E.1.

SITUATIONS VACANT

Assistant Design Engineers and Draughtsmen required for design and detailing of reinforced concrete structures. A range of appointments offer excellent prospects to both qualified men and those who are studying for qualifications.

Five day week. Superannuation Scheme. Apply stating age, qualifications and experience to :

Personnel Manager (A.D.E.3),
John Laing & Son Ltd.,
London, N.W.7.

ASSISTANT Designer/Detailers, Detailer/Draughtsmen and Trainee/Draughtsmen required for interesting work on varied types of reinforced concrete structures. Five day week. Luncheon Vouchers. Apply with usual details to John F. Farquharson & Partners, Chartered Structural Engineers, 34, Queen Anne Street, London, W.1. LANgham 6081.

CIVIL Engineering Designer required for Contractor's office in Westminster. Applicants age 26-32 must be graduates with mathematical ability and at least three years experience in design of Reinforced and Prestressed Concrete Structures, preferably also with some site experience. Written applications only, giving details of experience and qualifications to : Tileman & Co. Ltd., Romney House, Tufton Street, London, S.W.1.

CIVIL ENGINEERING DRAUGHTSMEN

required for design of reinforced concrete structures and foundations, steel structures and brick buildings in connection with heavy Engineering and Chemical Industry. Applicants should have relevant academic qualifications up to at least N.C. Standard and sufficient experience to enable them to prepare drawings from verbal and briefly sketched instructions. A knowledge of site surveying and levelling is also necessary.

Write for application form to Personnel Manager,
IMPERIAL SMELTING CORPORATION LTD.,
Avonmouth, Bristol.
Quoting Reference CD/SE.

**CIVIL ENGINEERING
and
STRUCTURAL ASSISTANTS**
(Intermediate and Junior)

required with design or detailing experience in any of the following :—

(a) Drainage and Sewage Purification Schemes.
(b) General Civil and Structural Work, including steel work or reinforced concrete.
(c) Hydro-Electric Works.
(d) Water Supply.
(e) Roads and Bridges.

Superannuation Scheme. Apply by letter stating age, qualifications and experience to Babtie, Shaw & Morton, Chartered Civil Engineers, 17, Blythswood Square, Glasgow, C.2.

CONSULTING Engineers have vacancy for Resident Engineer on interesting city development. Applicants should be qualified as position is one of responsibility. Bylander, Waddell & Partners, 169, Wembley Park Drive, Wembley.

CONSULTING Engineers have vacancies for Reinforced concrete detailers and designer/detailers in newly opened offices at Wembley Park. Salaries according to experience. Apply to Bylander Waddell & Partners, 169, Wembley Park Drive, Wembley, Middlesex.

CONSULTING ENGINEERS

have permanent pensionable (if desired) vacancies for experienced draughtsmen who are interested in advanced structural design, draughtsmanship and building techniques. The structures, large and small, in most cases have strong architectural interest as well and the materials used are as varied as the structures ; concrete, brick, steel and timber with strong emphasis on concrete, reinforced or stressed.

Applicants should have five to ten years drawing office experience and preferably some knowledge in and aptitude for design. Help will be given to those studying for professional examinations with opportunities (as they occur) to gain necessary site experience. Age between 23 and 35 with approximate basic salary range of £750 to £1,200.

Pleasant working conditions in friendly atmosphere in new modern offices. Five day week with good holidays (current as well). In addition to the basic salary, generous annual bonuses are paid rewarding efficiency and hard work directly related to individual effort. Luncheon vouchers, car allowance and travelling expenses between home and office are paid in addition to basic salary.

Please write or telephone for interview to :—

CHARLES WEISS & PARTNERS
123, King Street, Hammersmith
London, W.6.
Phone : Riverside 7843

Travelling (and hotel if necessary) expenses incurred for attending interview will be refunded.

CONSULTING Structural Engineers practice available to qualified man. Work to capacity in hand, useful conscientious staff, and compact site in Westminster area. Owner retiring. Scope for expansion, or would be useful take-over for over-worked practice. Write to Box 9086, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

D. W. COOPER requires designer/draughtsmen experienced in reinforced concrete and/or steelwork and/or timber engineering. Apply in writing to 165, Westmorland Road, Newcastle-upon-Tyne, 4. Stating age, qualifications and marital status.

DESIGNER/detailers and draughtsmen experienced in reinforced concrete required for interesting work on Nuclear Power Stations. Apply in writing to the Chief Engineer, Nuclear Civil Constructors, 52, Carnaby Street, London, W.1.

DESIGNER/detailers required with experience of reinforced concrete or structural steelwork. Work may range from structural frames to prestressed concrete bridges with scope for advancement but does not involve working outside London except for site visits. Assistance can be given towards A.M.I.C.E. or A.M.I.Struct.E. Write for appointments giving previous experience, Donovan H. Lee, Consulting Engineer, 66, Victoria Street, S.W.1.

W. & C. FRENCH LTD. urgently require Designer/Detailer, mainly for reinforced concrete design and detailing. Applicants must be keen to learn about new methods and types of construction. Apply in writing, stating age, salary required and full details of experience, to Personnel Manager, 50, Epping New Road, Buckhurst Hill, Essex.

FOUNDATIONS, Underpinning, Jacking. Structural engineers interested in this field who have a degree or the equivalent and two or three years post-graduate experience are invited to apply to Pynford Ltd., 74, Lancaster Road, London, N.4, who have vacancies in their steadily expanding design and supervision department.

SITUATIONS VACANT—continued

FULLY qualified Engineer to be directly responsible to the Partners required by medium sized firm of Consulting Engineers in Westminster. Should have had varied civil and structural engineering experience including road, bridge and railway works. Age 35 to 45. Write to Box 9088, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

FLEMING BROS.
(Structural Engineers) Ltd.,
invite applications for the position of
TECHNICAL DIRECTOR

Applications are invited for this important appointment which calls for a man of proven ability in the Structural Engineering field, who is fully conversant with modern developments in the design of Steel Structures.
Preferably, he should be qualified to A.M.I.Struct.E. level or a University Graduate and be confident about leading a team of first class Technicians in an expanding group of Companies. A generous salary and entry into a Top Hat Pension Scheme is offered.

Applications will be treated in the strictest confidence. These should be marked " Strictly Private," and addressed to :—

The Managing Director,
Messrs. Fleming Brothers,
(Structural Engineers) Ltd.,
Stanchion House,
1, Park Gardens,
GLASGOW, C.3.

G.K.N. REINFORCEMENTS LIMITED

have vacancies in their Birmingham Drawing Office for Reinforced Concrete Designers and detailers, also Draughtsmen and Draughtswomen who would have opportunities for training as detailers. Five day week, Pension Scheme, Staff Canteen. Write to :
Chief Engineer,

G. K. N. Reinforcements Ltd.,
Alma Street, Smethwick 40, Staffs.
Or
Phone Smethwick 1991.

JUNIOR Assistants wanted for consulting engineer's office. Apply in writing to W. E. J. Budgen & Partners, 54, Queen Anne Street, W.1.

LONDON W.1 and Cardiff Offices of Consulting and Civil Engineers require qualified senior designers and designer/detailers with reinforced concrete, precast, prestressed and steel experience. Five day week, superannuation scheme. Full details to Ecrofnier & Partners Ltd., 20, Harcourt House, 19, Cavendish Square, W.1 or telephone Langham 1493.

REINFORCED Concrete Designers and Detailers required in Berkhampsted office of Consulting Engineers. Apply, stating age, qualifications, experience and salary required, to Ritchie & Partners, Kitsbury Road, Berkhampsted, Herts.

REINFORCED Concrete Engineers and Contractors, Victoria, require a Chief Engineer to interview Architects and Engineers, and to control the drawing office in preparation of schemes and working drawings, etc. for various types of R.C. buildings, including multi-storey blocks of flats. Preference given to a qualified man and those earning less than £1,750 p.a. at present, need not apply, also Chief Estimator, those earning less than £1,500 need not apply. Subject to satisfactory service the posts are pensionable. Five day week. L.Vs. Holiday arrangements will be honoured. Apply giving full particulars, experience, qualifications and salary expected to Box No. DA.410, c/o Whites Ltd., 72, Fleet Street, E.C.4. All replies treated in the strictest confidence.

OVE ARUP & PARTNERS

Will shortly be opening design offices in the Croydon and Watford areas with vacancies for Engineers and Detailers experienced in Structural Work.
Salaries in accordance with experience up to £1,300 a year, luncheon vouchers, staff pension scheme.
Please write to :—

OVE ARUP & PARTNERS,
13, Fitzroy Street, W.1.

R.C. DESIGNER/detailers and detailers required in Hammersmith office of Consulting Engineers. High salaries and good prospects with interesting work. Five day week ; Pension Scheme ; Holiday arrangements honoured. Apply in confidence with full details of experience and salary required to Alan Marshall & Partners, Federal House, 2, Down Place, W.6. Tel. RIVerside 8771.

REINFORCED Concrete Specialist Firm requires Designer/Detailers. Interesting and varied work, luncheon vouchers, etc. Only experienced men need apply.—Write The Reinforced Concrete Steel Co., Ltd., 54, Victoria Street, S.W.1.

SENIOR qualified Design Engineer for Consulting Engineers, London Office, to take charge of R.C. section for blocks of offices, hospitals, etc. Write stating experience and salary required to Box 9087, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

STRUCTURAL and Civil Engineers require senior and intermediate designer-detailers experienced in either :
1. Reinforced Concrete or
2. Structural Steelwork.

Excellent opportunities in an expanding organization for the right men. Positions are pensionable and offer first class experience. Box No. 9075, STRUCTURAL ENGINEER, 43a, Streatham Hill, S.W.2.

SOUTH London Consulting Engineers require experienced reinforced concrete engineers, designer/draughtsmen and detailers. Applicants should have a minimum experience of three years in position applied for. Salary commensurate with experience and ability. Luncheon vouchers and Five day week. Please write giving details of age, qualifications and experience to Messrs. Leonard & Grant, 344, South Lambeth Road, S.W.8.

STAFF required for Consulting Structural Engineer's office. Senior and Junior Engineers and Detailers, experienced in R.C. framed structures. Five day week. L.V. Apply in writing to Felix J. Samuely and Partners, 231-233, Gower Street, London, N.W.1., stating age, experience and salary required.

FOR HIRE

LATTICE Steel erection masts (light and heavy), 30 ft. to 150 ft. high, for immediate hire. Bellman's, 21, Hobart House, Grosvenor Place, S.W.1.

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GUARANTEED Coaching for Inst. Struct. Engrs., Inst. Civ. Engrs., Inst. Mun. Engrs., etc. Study at home under highly qualified tutors with most successful school. Also courses in all aspects of Building, Engineering, Draughtsmanship, etc.—Write for Free book—International Correspondence Schools, Intertext House (Dept. 423), Parkgate Road, S.W.11.

SHOTBLASTING

SHOTBLASTING, Metal Spraying, Coating. Epikote, Araldite, P.T.F.E., P.T.F.C.E., Polythene, P.V.C., Neoprene and Hypalon coatings applied on site or at works. Loyne Ltd., Margaret Street, Ashton-under-Lyne, Lancs. Tel. ASH 4551/2/3.

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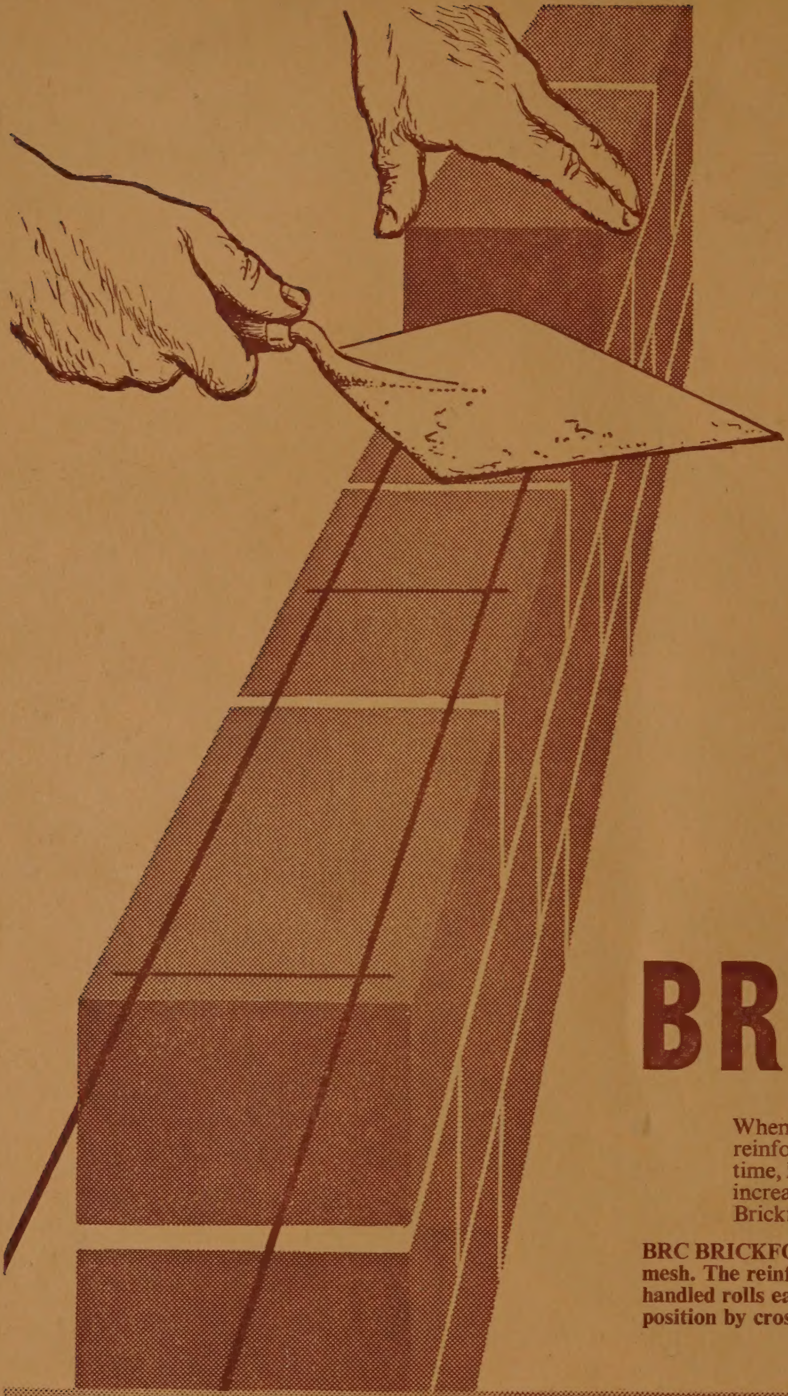
PELS Punching and Bar Cropping Machine for sale. Steel plate frame. Punches 1″ diameter through ¾″ plate. Depth of punch gap 20″. Crops round bars up to 1¼″ diameter ; flat bars up to 5″ x 1″. Hand or treadle stop motion. Arranged motor drive 400/440/3/50. Weight about 5 tons. F. J. Edwards Ltd., 359, Euston Road, London, N.W.1 or 41, Water Street, Birmingham, 3.



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